

National Park Service Jefferson National Expansion Memorial Gateway Arch

St. Louis, Missouri

Historic Structure Report – Volume 1

June 2010



Prepared by:

Bahr Vermeer Haecker Architects
Wiss, Janney, Elstner Associates, Inc.
Alvine and Associates, Inc.

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Prepared for:

National Park Service
Omaha, Nebraska

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Recommended:


Superintendent, Jefferson National Expansion Memorial

Date: 6/11/10

Concurred:


Associate Regional Director, Cultural Resources, Midwest Region

Date: 6.21.10

Approved:


Regional Director, Midwest Region

Date: 6-21-2010

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National Park Service Jefferson National Expansion Memorial Gateway Arch

St. Louis, Missouri

Historic Structure Report

EXECUTIVE SUMMARY

At the request of the National Park Service (NPS), Bahr Vermeer Haecker Architects (BVH) with subconsultants Wiss, Janney, Elstner Associates, Inc. (WJE), Alvine and Associates (Alvine) and BAF Consulting have prepared this Historic Structure Report (HSR) for the Gateway Arch at Jefferson National Expansion Memorial (JNEM) in St. Louis, Missouri.

The goal of the Historic Structure Report (HSR), as defined by National Park Service (NPS), is to serve as a critical planning and design document preparatory to the eventual execution of the ultimate treatment for the structure. Completion of the HSR is the first phase in this effort.

The purpose of the HSR is to provide a compilation of the findings of research, investigation, analysis, and evaluation of the historic structure. The preservation objectives for the historic property are identified and treatment measures recommended for implementing and accomplishing these objectives. The HSR serves as a basis for decision-making and direction for preservation of the structure. The report also functions as a record document of existing conditions and as a basis for planning future preservation and maintenance of the properties.

The national significance of the iconic Gateway Arch was recognized by its designation as a National Historic Landmark (NHL) in 1987. In the 1985 National Register

Nomination form prepared by Laura Soullière Harrison, the Arch is noted for its significance in the areas of architecture, engineering, and community planning. The October 2009 *General Management Plan for Jefferson National Expansion Memorial* (GMP) reinforced this assessment of the significance of the Arch while articulating a vision for the site that will guide future operations and management for the next fifteen to twenty years. The GMP provides a framework for resource management, visitor use, and development, and prescribes the desired future conditions of these resources and the visitor use environment. The chosen alternative—"Program Expansion"—has initiated an international design competition akin to the 1947 Memorial Competition. The intent of this new competition is to gain the greatest breadth of ideas for expanding interpretation, education opportunities, and visitor amenities at the Memorial. The impact of this effort upon the Arch has not yet been determined but is anticipated to include beneficial improvements to exhibits, heritage programs, barrier free access, and connections to the downtown St. Louis area.

As a structure considered primarily significant for its architectural design and engineering achievements, the period of significance for the Arch is associated with its initial design and construction. Therefore the HSR has identified an appropriate period of significance to be dated to the official dedication in May 1968. Further consideration is needed regarding the period of significance

for the visitor center, which includes both the lobby and museum. The visitor center was completed later and was excluded from the scope of this study.

The physical condition of the Arch is generally good in terms of both the interior and exterior. Some staining of the exterior skin has occurred since initial construction. This staining is difficult to assess completely without further materials studies. At the interior legs of the Arch, limited staining and surface corrosion are occurring to many steel components including bolt heads, plates, and stair assemblies. The micro-climate conditions at the interior of the Arch are unique and may be contributing to this corrosion. Interior occupied spaces, while essentially in their original configuration, have undergone some changes due to application of interpretive exhibitory materials (at the tram loading areas) and the replacement of worn wall and floor coverings (at the observation level). Mechanical and electrical systems vary in age and range in condition from good to poor. In terms of code compliance and accessibility, the Arch presents various challenges and limitations. Modification for full compliance may not be feasible, given the need to protect the character defining features of the structure.

This Historic Structure Report identifies a recommended treatment and scope of repair measures to address existing deterioration and future maintenance needs of the Arch based on conditions observed during the survey and through additional research. All of the recommendations have been developed in accordance with the Secretary of the Interior's Standards. Because the property's distinctive materials, features, and spaces are essentially intact and convey its historic significance without extensive repair or replacement; depiction at a particular period of time is not appropriate; and continuing use does not

require additions or extensive alterations, *preservation* is considered the appropriate treatment. Preservation is the act or process of applying measures necessary to sustain the existing form, integrity, and materials of an historic property.

Priorities for the treatments are as follows:

1. *Protection of Primary Structural Elements.* Studies and recommended further investigations and repairs as related to the protection of the primary Arch structure from deterioration should be undertaken. These are outlined in detail within the HSR.
2. *Life Safety and Functionality Upgrades.* Designs for appropriate life safety and functionality upgrades to the Arch should be studied and developed, with due consideration of the effect of any changes on the historic character-defining features of the Arch.
3. *Restoration.* Where altered, original interior finish materials and surfaces should be restored to a condition closer to the original design intent including materials, textures, and color.
4. *Cyclical Inspection and Maintenance.* In addition to the specific repairs recommended, cyclical maintenance tasks such as inspection, painting of exposed steel elements, cleaning, repair and /or replacement of finishes in the primary public areas of the Arch, and other ongoing maintenance tasks should be continually implemented to avoid damage to the historic building fabric and to reduce the need for large scale repair projects in the future.

INTRODUCTION

At the request of the National Park Service (NPS), Bahr Vermeer Haecker Architects (BVH) with subconsultants Wiss, Janney, Elstner Associates, Inc. (WJE), Alvine and Associates (Alvine), and BAF Consulting have developed this Historic Structure Report (HSR) for the Gateway Arch at Jefferson National Expansion Memorial (JNEM) in St. Louis, Missouri. The goal of the HSR is to develop planning information for use in the preservation of the Arch.

First developed by the National Park Service in the 1930s, HSRs are documents prepared for a building, structure, or group of buildings and structures of recognized significance to record and analyze the property's initial construction and subsequent alterations through historical, physical, and pictorial evidence; document the performance and condition of the structure's materials and overall physical stability; identify an appropriate course of treatment; and, following implementation of the recommended work, document alterations made through that treatment.

ADMINISTRATIVE DATA

Project Scope and Methodology

The purpose of the HSR is to provide a compilation of the findings of research, investigation, analysis, and evaluation of the historic structure. The preservation objectives for the historic property are identified and treatment measures recommended for implementing and accomplishing these objectives. The HSR serves as a basis for decision-making and direction for preservation of the building. The report also serves as a record document of existing conditions and as a basis for planning future preservation and maintenance.

The HSR addresses key issues specific to the Gateway Arch, including the construction chronology of the Arch; the existing physical condition of the exterior skin and structural systems; interior spaces and features; mechanical and electrical systems; code issues related to structural performance and public access to the Arch; and the historic significance and integrity of the structure.

Firm responsibilities were as follows:

- Bahr Vermeer Haecker Architects (BVH) – lead firm, project administration, project architects, accessibility and code review, and digital preservation
- Wiss, Janney, Elstner Associates, Inc. (WJE) – project historians, structural engineers, and architectural conservators
- Alvine and Associates (Alvine) – project mechanical, electrical, and plumbing engineers
- BAF Consulting – code consultants

The following project methodology was used for this study.

Research and Document Review. Archival research was performed to gather information about the original construction and past modifications and repairs to the Arch for use in assessing existing conditions and developing treatment recommendations. Documents reviewed included drawings, specifications, historic photographs, and other written and illustrative documentation about history, construction, evolution, and repairs to the subject buildings. The research for this study built upon the extensive historical and archival research performed by others, including Park Historian Robert Moore. Primary reference documents reviewed for this study included the following:

- Robert J. Moore, Jr., *Gateway Arch: An Architectural Dream* (St. Louis: Jefferson National Expansion Memorial, 2005).

- Laura Soullière Harrison, *National Register of Historic Places Nomination Form* (Washington, D.C.: National Park Service, 1985). This document was prepared in 1985 as part of the National Historic Landmark nomination of the Arch; NHL status was conferred in 1987.
- Regina M. Bellavia and Gregg Bleam, *Cultural Landscape Report* (Omaha, Nebraska: National Park Service, 1996).

Other reference documents and archival material used in development of this report are listed in the Bibliography. Also, the previous *Gateway Arch Corrosion Investigation, Part I*, completed in May 2006 by BVH and WJE, was reviewed and incorporated into the HSR.

The following archival repositories were visited or contacted in researching this study:

- NPS JNEM archives, St. Louis, Missouri
- NPS Denver Service Center archives
- Saarinen Archives, Manuscripts and Archives, Yale University Library, New Haven, Connecticut

Copies of selected archival documentation are provided in Appendix A. A description of research materials and sources reviewed and discovered is provided in the annotated bibliography within this report.

Two formal oral history interviews were performed as part of this study. An interview was conducted with Ken Kolkmeier, who worked as project engineer for the Pittsburgh-Des Moines Steel Company on the construction of the Gateway Arch, by Dan Worth of BVH, Stephen Kelley of WJE, and Robert Moore, NPS JNEM Historian. Al O'Bright, NPS Historical Architect, and Victoria Dugan of NPS JNEM also participated in the interview meeting. The interview was held at the JNEM Old Court House in St. Louis on January 14, 2009. At the

request of Mr. Kolkmeier this interview was not recorded; however, a summary of meeting notes was prepared and is provided in Appendix H.

An interview was also conducted with Bruce Detmers, an architect with Eero Saarinen and Associates who participated in the development of the construction documents and in site observation during construction of the Arch, by Deborah Slaton and Michael Ford of Wiss, Janney, Elstner Associates, Inc. The interview was held at the Yale University Saarinen Archives in New Haven, Connecticut, on April 1, 2009. This interview was recorded and supplementary notes were provided by Mr. Detmers. A transcript of the interview including accompanying notes is provided in Appendix H.

Condition Assessment and Documentation.

Concurrent with the historical research, a condition survey of the Arch was performed and observations documented with digital photographs, field notes, and annotation on baseline drawings. For purposes of the field survey, drawings were provided by the NPS. The condition assessment addressed the exterior and interior surfaces and features of the Arch and the adjacent tram load zones. In addition, the assessment addressed the structural systems within the legs of the Arch, which were examined from the stairs. The assessment also addressed the mechanical and electrical systems.

The visitor center and Museum of Westward Expansion were not included in this study, nor were approaches and entrances other than the tram load zones. The tram mechanism and capsules were also not included in this study, with the exception of architectural finishes. Landscape and site features were also not surveyed as part of this study, as the *Cultural Landscape Report* provides a comprehensive reference for the site and its components.

Materials Studies. In addition to the visual condition assessment, WJE performed laboratory finishes analysis of samples taken from selected interior spaces addressed in the HSR. All samples were viewed under reflected light microscopy following the procedures of ASTM D 1729, *Standard Practice for Visual Appraisal of Colors and Color Differences of Diffusely-Illuminated Opaque Materials*. Finish colors were assigned a Munsell color number. Additionally, color measurements of exposed coatings of selected elements were made in the field using a spectrophotometer. A complete discussion of methodology and the findings of this analysis are documented in Appendix B.

Development of History, Chronology of Construction, and Evaluation of Significance. Based on historical documentation and physical evidence gathered during the study, a context history, a detailed history of the Arch design and construction, and a chronology of construction were developed. An evaluation of the significance was also prepared, taking into consideration previous historical assessments including the National Historic Landmark (NHL) documentation and other reference documents, as well as guidelines provided by *National Register Bulletin 15: How to Apply the National Register Criteria for Evaluation*.¹ This evaluation of history and significance provided the basis for the development of recommended treatment alternatives.

Guidelines for Preservation. Based on the evaluation of historical and architectural significance of the structure, guidelines were

prepared to assist in the selection of preservation treatments.

Treatment Recommendations. The Secretary of the Interior's Standards for the Treatment of Historic Properties guided the development of treatment recommendations for the significant exterior and interior features of the Gateway Arch. Following the overall treatment approach of *preservation*, the specific recommendations addressed observed existing distress conditions as well as long-term preservation objectives.²

Preparation of Historic Structure Report. Following completion of research, site work, and analysis, a narrative report was prepared summarizing the results of the research and inspection and presenting recommendations for treatment. The HSR was compiled following the organizational guidelines of *NPS Preservation Brief 43: The Preparation and Use of Historic Structure Reports*, with modifications to organizational structure for purposes of this project.³

Drawing Digitization. As part of the HSR project scope, a large amount of original contract documents and other miscellaneous drawings and documents were digitally scanned to make them available to authorized HSR and NPS researchers. The drawings consisted mainly of diazo shop drawing prints and sepia prints that had been folded and stored in boxes. It was determined that this

¹ *National Register Bulletin 15: How to Apply the National Register Criteria for Evaluation* (Washington, D.C.: National Park Service, National Register of Historic Places, 1990, revised 1995).

² Kay D. Weeks and Anne E. Grimmer, *The Secretary of the Interior's Standards for the Treatment of Historic Properties with Guidelines for Preserving, Rehabilitating, Restoring & Reconstructing Historic Buildings* (Washington, D.C.: National Park Service, Historic Preservation Services, 1995).

³ Deborah Slaton, *Preservation Brief 43: The Preparation and Use of Historic Structure Reports* (Washington, D.C.: National Park Service, Technical Preservation Services, 2005).

information was not available in any other archive and that digital scanning of the drawings would facilitate access to the information while protecting the fragile documents. Therefore, scanning was completed as part of the HSR research process.

Also scanned during this process were other valuable documents found in the files of the JNEM maintenance office and others contained in the JNEM archives. These documents consisted of copies of original construction documents from Eero Saarinen's office, original design competition entry drawings, and various diazo prints of maintenance projects undertaken since the completion of the Gateway Arch. The digitized material included drawings for the Arch, railroad tunnel, floodwall, and site.

The initial conservation efforts were undertaken at the Corps of Engineers conservation laboratory in St Louis, Missouri, during the summer and fall of 2008. At the laboratory, each drawing was flattened, measured, and listed in a database. Following this process the drawings were transported to the Washington University West Campus Library, where digitization was completed.

The drawing and document digitization was performed by Digital Preservation of Chesterfield, Missouri, a firm specializing in digitizing land plats, records, maps, historic documents, and artifacts. The digitization process was appropriate to the fragile nature of the documents in that it did not require the documents to be fed through a traditional roll or flat bed scanner. Instead, a digital photographic process was utilized, including a large format digital 4 by 5 view camera to produce high resolution images. This process allowed documents to be placed flat upon a copy stand with the camera located above the document. The camera uses traditional photographic lenses, allowing depth of field to

be achieved so that all areas of the document are in focus. The density of each scan was approximately 300 dots per inch (DPI) for a 30 inch by 40 inch drawing, resulting in a digital file of around 762 megabytes (MB) per scan in tagged image file (TIF) format. All drawings that had no color were scanned in grey scale; those that had any color or were marked with either color pencil/ink or stamps were scanned in full color. All digital files were saved as TIF files and as portable document format (PDF) files and were loaded onto an external hard disk drive. This disk drive was then provided to the JNEM archives for their continued use in cataloguing and conservation. At the end of the digitization process, 3,218 drawings and documents had been digitized.

Examples of the scanned documents are provided in Appendices H and I.

Archival Document Summary

Team representatives visited the JNEM archives located at the Old Courthouse, which is the repository of many documents regarding the design and construction of the Gateway Arch. The investigation efforts included a limited amount of archival research of the Gateway Arch materials within the archives including past reports and studies, correspondence files, drawings, contract documents, etc. Materials reviewed also included government-supplied drawings from the NPS Denver Service Center archives including copies of the original contract documents of the Gateway Arch. The goal of this effort was to review these documents to help determine if any of the changes in construction techniques, materials, or methods might be contributing to, or would inform the team of what may be occurring on, the interior and exterior of the Arch in terms of corrosion. Due to the limited scope and the time available in the Part I investigation, many

documents were not viewed. The JNEM archives contain well over 1,000 boxes of materials as well as other materials housed in other types of storage. Many of the documents contained within the archives are not fully processed. Nevertheless, the team was able to review key correspondence files and drawing files that covered the period of time between opening of bids through and near the completion of the Arch construction.

The collections reviewed from the JNEM archives include:

- RU104 – Jefferson National Expansion Memorial Association records
- RU106 – Office of the Superintendent records
- RU122 – MacDonald Construction Company papers
- RU123 – Richard Bowser Collection
- RU131 – Ted Rennison papers
- RU134 – Shop Drawings, includes shop drawings for Arch erection, fabrication of steel and stainless steel Arch segments, crane construction, creeper supports, and the tram system as well as mill tests and samples.

A brief review of the archival documents revealed the following important events and issues, many of which have been cited and mentioned, namely:

- The Arch, during bidding and construction phases, underwent continuous modifications. The basic design criteria along with the modifications were challenged by various parties involved with the Arch design and construction, primarily in relation to relocation of the railroad tracks.
- Changes were made to the Arch segments both in the field and in the shop fabricating facilities until final completion.

- Welding and erection techniques were modified as new phases of construction were started.
- Discussions regarding issues of stainless steel protection during fabrication, erection, and after installation, including final cleaning, continued throughout construction.
- Issues of erection schedule and Arch closure were reviewed, debated, and contested beyond midway through the construction of the Arch. The closure force at the topping of the Arch calculated by the steel fabricator, Pittsburgh-Des Moines Steel Company, exceeded the allowable design limits estimated by the structural engineer. There was great concern about introducing additional moment forces that might cause stress and distortion in the exterior stainless steel skin.
- Problems with post-tensioning steel bars in the segments below 300 feet exacerbated the closure issue. Approximately twelve post-tensioning bars were abandoned early in the construction process due to interference of grout with filling of the post-tensioning tubes. Additional bars were added and additional closure thrust had to be incorporated due to these modifications. These modifications were finally accepted by the structural engineer.
- Additional closure stresses required additional diaphragm steel plating to be added at the segments above Segment 45.
- Unacceptable distortions occurred in the 1/4-inch stainless steel plate material during welding tests. Procedures, design, materials, and methods were changed to minimize the distortion. Distortion was still present in many of the completed segments.
- Segments 45 on both north and south legs were rejected after the north section was installed due to distortions in the stainless

steel skin caused by deforming the segment during installation. Fixes were prescribed, debated, and implemented for the installed north section. Both the north and south sections were filled with lightweight concrete along with other internal bracing modifications to help alleviate skin distortion.

- As the final segments were fabricated and installed (above Segment 45), distortion was minimized by improved quality and fabrication techniques and appearance was improved.

The following is a brief summary of information from the archival materials reviewed within the JNEM archives that relate to the issues noted above. While not inclusive of all relevant archival documentation, this information is representative of the issues, results, and discussions surrounding those items noted above. The participants in the correspondence described in this summary include staff of the NPS JNEM and the NPS Eastern Office of Design and Construction (EODC); the architect, Eero Saarinen and Associates (Saarinen and Associates); the consulting engineer, Severud Elstad Krueger and Associates (SEKA); the general contractor, MacDonald Construction Company (MCC); and the segment fabricator, Pittsburgh-Des Moines Steel Company (PDM).

- **August 1961 – Letter from Saarinen and Associates** summarizes the issue of closure force at the topping of the Arch and states that this force would introduce an additional moment of approximately 725 kips. (RU122, MCC papers, Box 9, Folder 4)
- **October 5, 1962 – Letter from SEKA** notes that excessive closure stresses may require additional reinforcing at segments above level 45. (RU122, MCC papers, Box 9, Folder 6)
- **November 6, 1962 – Letter from Saarinen and Associates** notes that the best method for closure would utilize a truss system near 500 foot level to stabilize Arch legs versus other methods including guys/cables which may overstress the concrete segments below 300 feet. (RU122, MCC papers, Box 9, Folder 6)
- **November 23, 1962 – Letter from SEKA** agrees with the truss method and additional jacking forces for Arch closure. (RU122, MCC papers, Box 9, Folder 6)
- **October 8, 1962 – Change Order No. 7** approves the use of stud bolts as a method to fasten Z-channel stiffeners to the stainless steel face plates to prevent the distortion that was caused by the originally specified method of welding stiffeners to the stainless steel plate. (RU122, MCC papers, Box 5, Folder 14)
- **March 1, 1963 – Letter from Saarinen and Associates** requires additional steel diaphragms above Segment 45 in response to a revision of the structural analysis. (RU122, MCC papers, Box 9, Folder 9)
- **March 29, 1963 – Job site meeting memo summary** records a discussion of protection and cleaning of stainless steel. Segment 71 was selected for demonstration and a final plan for cleaning and protecting steel was to be developed after testing. (RU122, MCC papers, Box 9, Folder 9)
- **May 9, 1963 – Memo of meeting between PDM, MCC, Saarinen and Associates, and SEKA**, in which PDM claims that the design of the Arch is deficient and that buckling will occur in areas above 300 feet. Saarinen and Associates states that no design deficiencies are contained in current design. (RU122, MCC papers, Box 10, Folder 1)
- **August 23, 1963 – Letter from Saarinen and Associates** notes streaking at the south ribs of the Arch segments. Saarinen and Associates and JNEM request

- information regarding the erection procedure and cleaning procedure as promised by MCC. (RU122, MCC papers, Box 10, Folder 2)
- **November 13, 1963 – MCC letter** reports that post-tensioning steel bars at Segment 61 cannot be tensioned as intended due to obstructions caused by grouting at the tubes. (RU122, MCC papers, Box 10, Folder 4)
 - **November 14, 1963 – JNEM memo to MCC** expresses continuing concern over unsatisfactory practices of protection of stainless steel surfaces. LeRoy Brown of JNEM again requests information and submittals. (RU122, MCC papers, Box 10, Folder 4)
 - **November 20, 1963 – Letter from SEKA** notes that closure crown thrust of 850 kips exceeds allowable design and will cause issues at the extrados of the Arch. (RU122, MCC papers, Box 10, Folder 4)
 - **November 29, 1963 – MCC Erection calculation** is submitted with closure forces and truss system. (RU122, MCC papers, Box 10, Folder 4)
 - **December 10, 1963 – Letter from MCC** states that an additional 80 kips for crown thrust will be needed. MCC requests Change Order 24 to approve additional post-tensioning bars and revised crown thrust. (RU122, MCC papers, Box 10, Folder 5)
 - **December 11, 1963 – Conference notes from Bruce Detmers** record a discussion regarding crown thrust calculations. Saarinen and Associates notes that correcting the deficient post-tensioning steel bars at Segment 61 will increase the crown thrust. (RU122, MCC papers, Box 10, Folder 5)
 - **December 11, 1963 - Letter from MCC** contains the stainless steel cleaning and protection procedures submittal with a four foot square sample for approval. (RU122, MCC papers, Box 10, Folder 5)
 - **February 19, 1964 – Letter from the EODC to JNEM Park Superintendent** states that the welding of stiffener plates at Segment 45 is causing distortion of the stainless steel plate surface and that this appearance is unacceptable to EODC. (RU122, MCC papers, Box 11, Folder 2)
 - **February 19, 1964 – Letter from JNEM to MCC** requests that access panels be provided at the upper Arch segments to insure welding inspections at Segment 45 and above. (RU122, MCC papers, Box 11, Folder 2)
 - **February 26, 1964 – Memo from JNEM to EODC** states that distortions in the stainless steel plates are caused by welding. Two possible scenarios are discussed for proceeding with the project: 1) accepting the distortions; or 2) issuing a change order for stud welding as a means of attaching stiffeners to the stainless steel plate. (RU122, MCC papers, Box 11, Folder 2)
 - **March 19, 1964 - Letter from MCC to JNEM** requests eliminating the access panels. MCC also requests the use of plug welding in lieu of high strength bolts above Segment 45 and the substitution of Z-stiffeners in lieu of steel angles to stiffen the stainless steel face plates. MCC states that the result would be minimal visual effects due to welding distortion. (RU122, MCC papers, Box 11, Folder 3)
 - **April 3, 1964 - Letter from MCC to JNEM** states that the stainless steel inspection specification submittal and test panel has been approved by the resident architect. [Note: it is unclear in the source document if this refers to Robert E. Smith, Chief Architect of the EODC at the time, or another architect.] (RU122, MCC papers, Box 11, Folder 4)
 - **April 16, 1964 - Letter from MCC to JNEM** confirms that there will be no cost changes for revisions described in the

- March 19, 1964, letter. (RU122, MCC papers, Box 11, Folder 4)
- **May 5, 1964 – Change Order 30** is an approval with no cost adjustment for revising the welding stiffening scenario of segments above Segment 45. (RU122, MCC papers, Box 6, Folder 15)
 - **June 9, 1964 – Letter from PDM to MCC** notes a concern about aerodynamic forces. PDM has by this time retained Hanson Holley & Briggs, structural and aerodynamic engineers from MIT, to study aerodynamic forces on the Arch. PDM notes that further study is needed. (RU122, MCC papers, Box 11, Folder 6)
 - **June 22, 1964 – Press release from National Park Service** notes stoppage of work at Segment 45. The press release summarizes issues related to the change of structural type from concrete to steel at the 300 foot level and questions regarding structural stability that have been raised by PDM. (RU106, Office of the Superintendent Records)
 - **August 24, 1964 – Temperature testing sensors** have been located at Segments 44 and 45. (RU122, MCC papers, Box 11, Folder 8)
 - **August 29, 1964 – Meeting minutes from site meeting by MCC** discuss deformed panels at Segment 45 observed at north and south legs of the Arch. The panels were warped due to stresses induced by cables during fitting of these segments to the ones below to align for welding. The minutes note that north Segment 45 is already welded in place and that south Segment 45 is still on the ground. Both segments have been rejected by Saarinen and Associates and JNEM. (RU122, MCC papers, Box 11, Folder 8)
 - **October 3, 1964 - Letter from MCC to Saarinen** suggests that Segments 45 (both north and south) also be filled with lightweight (3,300 psi) concrete at 110 pounds per cubic foot to assist with correction of the deformed panels. (RU122, MCC papers, Box 12, Folder 2)
 - **October 29, 1964 – Letter from PDM to MCC** notes that secondary steel members have been installed to improve flatness. (These secondary steel members are the wide flange members visible on the interior of the Arch above Station 45.) PDM agrees with MCC's recommendation to use lightweight concrete fill within Segment 45 to assist with improving the panel flatness. PDM also notes that fabrication of Segment 44 will be utilizing these methods and that they will request approval before other segments are fabricated. (RU122, MCC papers, Box 12, Folder 2)
 - **November 6, 1964 – Change Order 34** approves use of turnbuckles to help eliminate warping in Segment 45 (north and south) and also use of lightweight concrete fill. (RU122, MCC papers, Box 6, Folder 19)
 - **January 14, 1965 – Change Order 30** approves a time extension of seventy-five days beyond the original contract. Plug welding is approved to be utilized at segments above Segment 45. Z-stiffeners are also approved as a means to eliminate warpage of stainless steel panels. The elimination of inspection panels in all segments above Segment 45 is also approved. (RU122, MCC papers, Box 6, Folder 15)
 - **November 22, 1966 – Letter from JNEM to MCC** regarding Change Order 46 reduces the contract price by \$367,631.20 to compensate the government for damages to the Gateway Arch stainless steel surfaces caused by erection procedures, shipping, and cleaning. (RU122, MCC papers, Box 15, Folder 3)

PART 1 – DEVELOPMENTAL HISTORY



Jefferson National Expansion Memorial Gateway Arch. Source: HABS photograph by Jack Boucher, 1986.

ST. LOUIS, GATEWAY TO THE WEST

Throughout its history, St. Louis has defined itself as the “Gateway to the West.” Located fifteen miles south of the confluence of the Mississippi and Missouri Rivers, St. Louis was established by Pierre Laclède in 1764 as a French fur-trading post. The site had a high limestone bluff extending approximately two miles along the river which provided a suitable location for a settlement protected against flooding.

In 1803 during Thomas Jefferson’s presidency, the Louisiana Territory including the village of St. Louis was purchased from France, nearly doubling the size of the United States. In the years following the Louisiana Purchase, the western frontier remained a place for mountain men, fur-trappers, and explorers. St. Louis was a major post where frontiersmen could sell their goods or acquire supplies before venturing further west. In 1809, the town of St Louis was incorporated.

The development of the steamboat fueled St. Louis’ success as an inland port economy. In 1817, the *Pike* was the first steamboat to arrive in St. Louis, introducing the city to commercial steamboat commerce. During the decades that followed, St. Louis was at the crossroads of steamboat traffic. The Missouri River linked the city to the western frontier. The Ohio River, running primarily east-west, proved an effective thoroughfare that directly connected St. Louis to eastern markets and extended to urban centers in the northeast. The Mississippi River provided a north-south backbone for the river network, giving access from the northern frontier of Minnesota as well as downriver to markets in New Orleans and the waters beyond. By 1850, St. Louis was the second largest port by tonnage in the United States, exceeded only by New York.

After incorporation as a city in 1822, St. Louis’ trade-based success prompted the

restructuring of the commercial riverfront with more permanent buildings. One-story brick commercial structures were built and the old town and the city began to expand west and north. In 1828 a brick courthouse was constructed, and six years later a stone Roman Catholic Cathedral was built to replace an existing brick church.⁴ By 1850, St. Louis was the largest city west of Pittsburgh. It was the center of steamboat traffic on the Mississippi River as well as the terminus for stage coach lines from the east and the unofficial starting point for pioneer trails heading west. St. Louis had established itself as a gateway to the western frontier (Figure 1).

The Civil War marked the transition of St. Louis from a river boat city to a railroad hub. Although St. Louis was outside the area of direct Civil War conflict, St. Louis riverboat commerce relied on its relationship with southern markets. Thus, the Union blockade on trade with the Confederate states devastated the St. Louis economy and nearly ended steamboat traffic on the Mississippi.

St. Louis became a river-crossing point for rail traffic. Railcars reaching St. Louis were loaded onto ferry boats and transported one-by-one across the river, where they were reconnected to locomotives and continued on their journey. The absence of a railroad bridge across the Mississippi River forced train traffic to stop in St. Louis. Finally in 1874, the Eads Bridge was dedicated and became the first bridge in St. Louis to span the Mississippi River. In 1889, entrepreneur Jay Gould orchestrated an agreement between six of the St. Louis railroad companies to form the Terminal Railroad Association (TRRA).

⁴ The Roman Catholic Cathedral is currently referred to as the Old Cathedral and remains at its original location within JNEM.



Figure 1. Old St. Louis in 1855. Lithograph from Yale University Archives, Eero Saarinen Collection, Manuscript Group 593.

The organization would manage the Eads Bridge and thus secure an efficient and economical means of transporting passengers and freight across the river.

In the second half of the nineteenth century, the development of the railroads accelerated and steamboat traffic decreased. The shift in the economy was nowhere more apparent than in the downtown business district of St. Louis. Prior to the Civil War, the waterfront area had been the center of commerce. The new dominance of the railroad encouraged the westward movement of the urban core to higher ground away from the narrow streets and decaying industrial buildings of the riverfront. The historic city core was left to industrial and warehouse uses. The river's edge was dominated by railroad traffic navigating its way across the Mississippi

River bridges. By 1930, there were five railroads along the St. Louis levee.

As the country expanded and transportation and technology improved, the role of St. Louis as the link between the east and the west evolved and was reflected in its built environment. The development and role of St. Louis in the expansion of the country is illustrated by its historic riverfront district and memorialized and symbolized through Eero Saarinen's Gateway Arch.

JEFFERSON NATIONAL EXPANSION MEMORIAL

Initiated in the Depression era, the Jefferson National Expansion Memorial (JNEM) project was first proposed as a means of rejuvenating the St. Louis riverfront and providing economic relief to the city. The project rapidly achieved national attention and public support, concurrent with the development of financial difficulties and legislative disputes. After four decades of debate, controversy, and delays the memorial was completed, culminating in a monument that not only commemorated the vision of Thomas Jefferson and the struggles of the traders, frontiersmen, and pioneers but also the determination and persistence of individuals who were instrumental in the development of the national historic site. The history of JNEM is more fully described in *Administrative History: Jefferson National Expansion Memorial National Historic Site*, written by Sharon A. Brown.⁵

Creation of a National Historic Site

The idea for a St. Louis memorial commemorating the Louisiana Purchase had been discussed as early as 1887, when James G. Blaine suggested building a statue along the riverfront. However, it was the dedication of St. Louis attorney Luther Ely Smith that led to the establishment of JNEM. In late 1933, upon returning to St. Louis by train, Smith was troubled by the appearance of the decaying historic riverfront district and felt that the creation of a monument could bring economic development, provide work relief, and revitalize the historic waterfront area.

Smith's idea for a monument was well received by St. Louis Mayor Bernard

Dickmann. In April 1934, the Jefferson National Expansion Memorial Association (JNEMA) was organized. The title of the association reflected a focused direction for the project as a monument of national scope that would commemorate the vision of Thomas Jefferson and the sacrifices of pioneers in opening the West. JNEMA, under the guidance of a determined Smith and with the political savvy of Dickmann, became the driving force in obtaining support, soliciting funding, and developing a memorial plan. St. Louis architect Louis LaBeaume was hired to establish the site boundaries, define the design parameters, and outline his concept for a national design competition. Early in the process, a consensus was reached to raze the majority of the warehouse and industrial buildings in the historic St. Louis riverfront district.

In June 1934, the U.S. Congress established the fifteen-member United States Territorial Expansion Memorial Commission to oversee the feasibility of a national monument in St. Louis. On April 13, 1935, the anniversary of Thomas Jefferson's birthday, the commission's executive committee, having reviewed the progress made by JNEMA, approved the plan for the memorial. This plan included a national design completion, commemoration of events of both national and local historical significance, and an estimated budget of \$30 million for land acquisition, development, and planning.

On September 10, 1935, a St. Louis city bond issue to partially fund the memorial was passed by voters. The city was prepared to contribute up to \$7.5 million to the construction of the memorial, with one dollar contributed by the city for every three dollars contributed by the federal government.

⁵ Sharon A. Brown, *Administrative History: Jefferson National Expansion Memorial National Historic Site* (Washington, D.C: National Park Service, June 1984).



Figure 2. Aerial view of the St. Louis riverfront and downtown, 1930s. The Eads Bridge is at the extreme right edge of the photograph. Source: JNEM archives, image V106-4822.

At the federal level, the decision was made to designate the project location a national historic site to allow for federal funding of construction and future maintenance. Executive Order 7253, signed by President Franklin D. Roosevelt on December 21, 1935, made JNEM the country's first National Historic Site under the Historic Sites Act.⁶ The land would serve as

... a permanent memorial to the men who made possible the territorial expansion of the United States, particularly President Thomas Jefferson and his aides, Livingston and Monroe, who negotiated the Louisiana

Purchase, and the hardy hunters, trappers, frontiersmen, and pioneers and others who contributed to the territorial expansion and development of the United States of America.⁷

The Executive Order authorized the NPS to acquire thirty-eight city blocks encompassing the site of Old St. Louis and to develop and preserve the site as JNEM. Within the site, 40 percent of the buildings were unoccupied in 1936. Given the decayed state of the

⁶ Historic Sites Act of 1935, 16 USC 461 to 467.

⁷ Brown, Chapter 1, 2, citing Pro Forma Decree of Incorporation of Jefferson National Expansion Memorial Association, June 11, 1934, JNEMA.

neighborhood, the NPS acquired the land by means of condemnation as opposed to purchase. By September 1938, review of all properties within the historic site boundary had passed through the courts and all of the buildings were under condemnation. Legal processes surrounding condemnation continued until January 27, 1939, when the United States Circuit Court of Appeals declared the federal government's attempts to condemn the land as a valid delegation of its legislative power. On June 14, 1939, federal funds totaling \$6.2 million, the entirety of the contracted land agreements, were dispersed to property owners.

While early efforts focused on the acquisition of land, the NPS started preliminary work on the interpretation of the site based on its concept of a memorial to Thomas Jefferson's vision of territorial expansion.

In 1936, the Old Courthouse was a vacant and dilapidated structure sited just outside the proposed boundary of JNEM. The Greek Revival-style building was constructed beginning in 1839 and had a Greek cross plan with Classical Revival style dome. By 1851, a new east addition was constructed, and a cast iron Italian Renaissance Revival dome was constructed in 1862. The building served as the county courthouse until 1930, when it was decommissioned and became vacant. The Old Courthouse displayed significant architectural merit as an example of Greek Revival-style civic architecture as well as historical interest as the site of two influential court cases regarding discrimination and human rights. In 1847 and 1850, the courthouse was the focus of debate as the site of the Dred Scott case. The pivotal law suit tested the rights of slaves who had once resided in free territories to seek their own freedom. In 1872, the courthouse was once again the center of political debate when Virginia Minor sued the St. Louis ward registrar for refusing her the right to vote in

the 1872 presidential election. The results of both cases favored the defendants but were influential in bringing the issues of personal liberties to the forefront of the American conscience.

On July 1, 1937, the City of St. Louis, understanding the severe state of disrepair and associated costs necessary in the restoration of the Old Courthouse, offered to transfer ownership of the building to the federal government. Federal acquisition of the courthouse offered the opportunity to contribute to the overall mission of the historic site. In addition, the building could serve as a suitable museum and office space for the NPS. In 1940, the land was deeded to the federal government. Federal funds were made available for the restoration of the courthouse.

The Mississippi River was an integral part of the history of St. Louis as a gateway to the western territory. Thus, the success of any memorial commemorating national expansion would depend on its relation to the waterway. For decades, St. Louis had thrived as a hub for railroad traffic. By the 1930s, three surface and two elevated tracks had been built on the levee and dominated the riverfront. The railroad tracks defined the eastern boundary of the memorial site and separated it from the riverfront. In August 1938, St. Louis Board of Public Service President Baxter Brown submitted a plan for relocation of the railroad tracks. The proposal combined a new tunnel to conceal the relocated tracks and re-grading of the site to elevate it over the tunnel. These modifications would eliminate the elevated and surface tracks and open up the views to the river.



Figure 3. Demolition of buildings on the memorial site, circa 1940. Source: JNEM archives, image V106-501a.



Figure 4. HABS photograph by Alexander Piaget of the Old Rock House, corner of Wharf and Chestnut Streets, circa 1933.

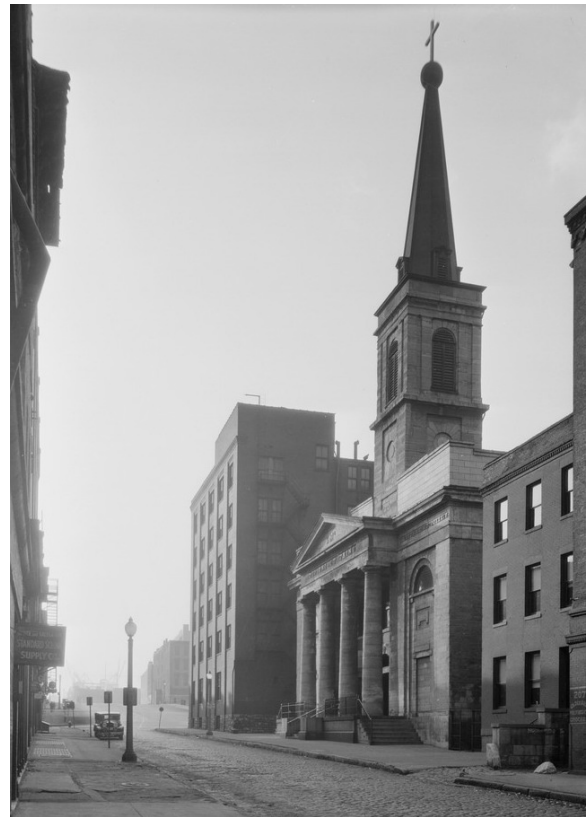


Figure 5. HABS photograph by Alexander Piaget of the Old Cathedral in its urban context, Walnut Street near Third Street, April 9, 1934.



Figure 6. View of completed demolition of the site, 1942. The railroad viaduct remained, separating the memorial site from the river front. Source: JNEM archives, image V106-5190.



Figure 7. Aerial view of the memorial site, October 1946, U.S. Navy photograph. Source: JNEM archives, image V106-4844. The Old Cathedral is visible near the center of the memorial site, while the Old Rock House remains near the river's edge adjacent to the railroad viaduct.

On October 10, 1939, the first signs of visible progress on JNEM were made as Mayor Bernard Dickmann initiated the demolition process with the removal of three bricks from an abandoned warehouse building. Demolition of most of the buildings on the memorial site was completed by 1942. A few buildings considered to be of historic interest were not demolished, including the Old Cathedral (which was the only building in the memorial site not acquired by the federal government) and the Old Rock House, an 1818 stone warehouse built by fur-trader Manuel Lisa at

the corner of Wharf and Chestnut Streets.⁸ The Old Rock House was “restored” with major alterations in 1939–1942 by the NPS. Historic American Buildings Survey documentation of many of the structures on the memorial site was completed prior to demolition.

Designing and Financing the Arch

The long tenure of JNEM Superintendent Julian Spotts from 1940 to 1959 was characterized by two significant events that shaped the development of the memorial. First, JNEMA sponsored a national design

⁸ Portions of the building may have been built as early as 1767.

competition that captured the imagination of the public. The resulting winner, Eero Saarinen, created a simple yet dramatic design that was both commemorative and inspirational. Second near the end of Julian Spotts' superintendency an agreement was made between the City of St. Louis, TRRA, and the NPS. The long awaited compromise was followed by the authorization of federal funding and extensive preparations for the first phase of construction on the Gateway Arch.

During World War II, progress on the JNEM site was limited. The Works Progress Administration (WPA) initiated the restoration of the Old Rock House and Old Courthouse buildings. However, efforts to establish a memorial were suspended as the country focused funds and attention on the war.

The Competition

Since its inception, JNEMA had planned to sponsor a national design competition for a suitable memorial, "transcending in spiritual and aesthetic values."⁹ The competition was officially announced in January, 1945, despite the fact that the funds required to stage the competition had not yet been raised and the United States Territorial Expansion Memorial Commission had not given approval. The latter proved most crucial, as success of the competition balanced on the assurance that the winning designer would be authorized to proceed with the design.

Guidelines for the memorial design were outlined by LeBeaume and submitted to the United States Territorial Expansion Memorial Commission for consideration. The design parameters focused on providing a fitting memorial while invigorating the riverfront and developing a setting integral with the

downtown community. LeBeaume's requirements included the building of an architectural memorial, preserving the site of Old St. Louis through a museum and reconstruction of Old St. Louis buildings, creating a living memorial to Thomas Jefferson, accentuating recreational opportunities, providing access to parking, relocating railroad tracks, and accommodating a new interstate highway. On May 28, 1947, two days before the competition opened, the parameters of the competition were endorsed by the commission.

In August 1946, George Howe, the Philadelphia architect responsible for the PSFS Building and other influential American modernist structures, was recruited by Luther Smith to be the professional advisor for the competition. The competition jury consisted of seven members; S. Herbert Hare, Fiske Kimball, Louis LeBeaume, Charles Nagel, Jr., Richard Neutra, Roland Wank, and William Wurster, many of whom had sensibilities influenced by the International style of architecture. Howe's appointment, combined with the assignment of modernist architects to the jury, was an indication of the desired design aesthetic for the memorial.

The two-stage contest consisted of an initial review by judges at which time five finalists were selected. The finalists were then given \$10,000 and three months to develop the second stage of design. Throughout the process, the identity of the competitors was kept secret. The winner would be selected following the second stage of design and be determined by secret ballot.

By the September 1, 1947, deadline, 172 architects and engineers had submitted designs for consideration. Entry No.144, Eero Saarinen's design, was given much praise as a beautiful and inspired design; Charles Nagel described the design as "an abstract form

⁹ Brown, Chapter 4, citing a Report, Smith to Jefferson National Expansion Memorial, November 4, 1944, JNEMA.

peculiarly happy in its symbolism.”¹⁰ However, criticism arose in regard to its practicality (Figure 8).

Entries for the second stage of competition were due February 17, 1948. Upon first review of the designs, the jury submitted votes and unanimously selected Eero Saarinen as the winner. The selection was announced on February 18 by JNEM; however, it was not until May 25, 1948, that the United States Territorial Expansion Memorial Commission voted to recommend the design for approval by the Department of the Interior and Congress (Figure 9).¹¹

Eero Saarinen was born in Finland in 1910 and immigrated to the United States with his family in 1923. His father, renowned architect Eliel Saarinen, was the first president of the Cranbrook Institute of Architecture and Design in Bloomfield Hills, Michigan. After studying sculpture in Paris and Architecture at Yale University, Eero Saarinen joined his father’s firm in 1937.¹² Saarinen’s entry into the JNEM competition combined his sculpture background and architecture education, characteristics which would become the trademark of his designs.

The Saarinen design consisted of the central Arch with a tree-lined mall and arcade. Saarinen’s catenary arch was derived from his initial concept of a three-legged structure.¹³

He was intent on using a simple iconic form, characteristic of the Jefferson Memorial or Washington Monument, realized in modern materials.



Figure 8. Photograph of the scale model of Saarinen’s competition entry. Yale University Archives.



Figure 9. Saarinen (shaking hand on the left) accompanied by (left to right) J. Henderson Barr, associate architect; Alexander Girard, painter; Dan Kiley, landscape architect; and Lily Saarinen, wife and sculptor, awarded first prize by William Wurster (far right) at the JNEMA banquet on February 18, 1948. Image courtesy of Yale University Archives, Eero Saarinen Collection.

¹⁰ Brown, Chapter 4, citing Charles Nagel, Jr., *A Sketch Report of the Jury Proceedings, Jefferson National Expansion Memorial*, September 23–26, 1947, JNEMA.

¹¹ Brown, Chapter 4.

¹² Laura Soulli re Harrison, *National Register of Historic Places Nomination Form: Gateway Arch* (Washington, D.C.: NPS, 1985). This National Register nomination serves as documentation for the National Historic Landmark listing of the structure.

¹³ A *catenary* is the curved shape assumed by a heavy cord or chain suspended from two points. The shape of the Arch is based on an inverted catenary.

Saarinen's design was used to generate support and excitement for the JNEM park. The inspirational design was well received by critics, with limited dissent among the general public. Some St. Louis residents referred to it as a "stupendous hairpin" or "stainless steel hitching post."¹⁴ The most severe criticism came from Gilmore Clarke, Chairman of the National Commission on Fine Arts, who perceived a resemblance between the design for the Arch and an exhibition structure in Rome, proposed under Mussolini in 1942.¹⁵

Railroad Relocation

Upon completion of the competition, attention was redirected toward the difficult and arduous task of resolving the dispute between the City of St. Louis, the NPS, and TRRA over the relocation of the railroad tracks.

The TRRA and the City of St. Louis were in favor of a Levee-Tunnel plan that placed the tracks along the riverfront. Saarinen and the NPS objected to this proposal, as it would obscure the relationship between the Arch and the water. Saarinen supported a Hill-Tunnel plan, which positioned the tracks on the west end of the site.

On December 6, 1949, the negotiating parties found common ground. After much debate and compromise, the parties agreed on a plan defined in a Memorandum of Understanding.¹⁶ Drafted by Saarinen, the plan outlined the removal of the five existing tracks along the levee and replacement with three tracks, one owned by the Missouri Pacific Railroad and two by the TRRA. The railroad lines were to

be positioned in a tunnel, no larger than 3,000 feet long and eighteen feet tall, approximately fifty feet west of their existing elevated location. The agreement was effective pending approval of the Missouri Public Service Commission (MPSC) of the lower-than-recommended tunnel height.¹⁷ A concession of the new plan was the demolition of the Old Rock House.

The successful compromise obtained through the Memorandum of Understanding was met with resistance. In 1950, efforts to secure Congressional funding were stalled until provisions were made for the relocation of the railroad tracks. Progress toward development of JNEM was further delayed by the start of the Korean War. During the war period, government spending was restricted and attempts to appropriate funds were temporarily halted. The future of JNEM was further compromised by the death of Luther Ely Smith on April 2, 1951. Smith had founded and directed the Association in its efforts to commemorate Jeffersonian Expansion.

Following the conclusion of the Korean War in 1953, and despite a lack of funds, in 1954 Congress authorized the appropriation of \$5,000,000 for construction of Saarinen's Memorial.¹⁸ Although no funds were immediately available, the act symbolized the support of the federal government in the project. In 1956, an additional \$2,640,000 was allocated to the JNEM project for the relocation of the elevated railroad tracks.¹⁹ Allocation of these funds was the first step to preparing the site for the construction of the Arch.

¹⁴ Regina Bellavia, historical landscape architect and Greg Bleam, landscape architect consultant, *Cultural Landscape Report for Jefferson National Expansion Memorial* (Omaha, Nebraska: Government Printing Office, 1996)

¹⁵ Ibid.

¹⁶ Brown, Chapter 5

¹⁷ Approval was given by MPSC on August 7, 1952.

¹⁸ Public Law 361 (H.R. 6549) May 17, 1954. The law specified the expenditure of funds on the Arch itself.

¹⁹ Brown, Chapter 6

On September 7, 1958, the determination of the JNEMA, the patience of the City of St. Louis and the NPS, and the inspiration of Saarinen were rewarded when President Eisenhower signed legislation amending the 1954 authorization to provide for the construction of JNEM in its entirety. A total of \$17,250,000 was allocated for construction.

Structural Design

Following the appropriation of funds, renewed excitement and energy surrounded the JNEM project. Eero Saarinen and Associates generated construction documents for the development of the levee and refinements were made to the design of the Arch and surrounding landscape.

Saarinen focused on developing the correct proportion and scale for the Arch, to achieve the desired iconic appearance, as well as the required structural stability throughout the construction process. Saarinen had originally envisioned a 590-foot-tall Arch, but as the St. Louis skyline increased in height, so did the Arch. By 1959, a 630-foot-tall Arch was planned with a width equal to its height (Figure 10).²⁰ Saarinen consulted Fred Severud, his long-time structural engineer, and Hannskarl Bandel of Severud, Elstad, Kreuger Associates of New York City to develop a structural solution to capture Saarinen's refined vision.

Fred Severud was an innovative civil engineer who had immigrated to the United States from Norway. Severud had worked on the Raleigh Coliseum, Madison Square Garden, and the Yale Hockey Rink, developing some of the first cable-supported roof structures in the United States. Severud believed that the engineer's responsibility was to use his background and inherent knowledge of

structure to develop problem-solving ideas, not just to perform calculations.²¹

Hannskarl Bandel, Severud's chief engineer, worked closely with him on the Arch design. Bandel was raised in Germany by a father who was an architect and a mother whose family owned the Bechtel Construction Company. Before immigrating to the United States after World War II, Bandel had gained experience as an engineer in the steel industry.

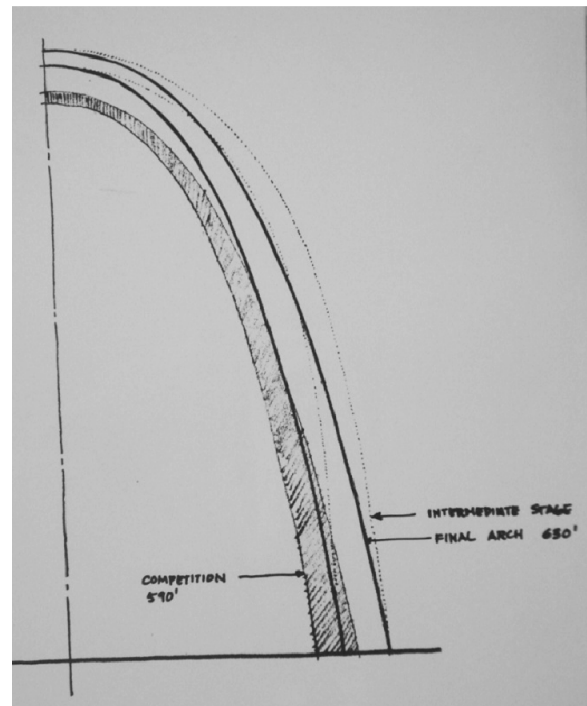


Figure 10. Evolution of the Arch height and shape from competition to final design. Courtesy of Yale University Archives, Eero Saarinen Archives.

²⁰ Saarinen sketches from Yale University Archives. Record Group 593; Series IV; Box 97.

²¹ Richard G. Weingardt, *Engineering Legends: Great American Civil Engineers* (Reston, VA.: American Society of Civil Engineers, 2005).

The structural concept for the Arch was a collaborative effort between Severud's and Saarinen's offices. During the design competition, Saarinen indicated that the Arch would be a steel structure filled with concrete. Severud introduced orthotropic design principles, which were new for the period.²² Following these principles, the Arch structure was designed to be supported by its skin. A carbon steel inner shell and stainless steel outer shell were set at slightly different weighted catenary curves and connected through stiffener plates. The interstitial space between these shells was filled with post-tensioned concrete at the lower half of the Arch.²³ The two interconnected skins thus helped support each other.

Bandel was responsible for reproducing Saarinen's soaring catenary shape in the structural design. When Saarinen tried to demonstrate his intent with a chain suspended in his hands, he could not achieve the slightly elongated effect he wanted. Bandel replaced some of the constant-sized links in the chain with variable links, thus changing the weight, its distribution, and the resulting shape—a weighted catenary. Saarinen then modified the design of the Arch through scale models and weighted catenary studies. The Arch structure developed as an equilateral triangle cross-section that measured 54 feet across at the base, tapering to 17 feet across at the top (Figure 11 and 12).²⁴



Figure 11. Eero Saarinen with several study models, circa 1959. Courtesy of Yale University Archives, Eero Saarinen Collection.

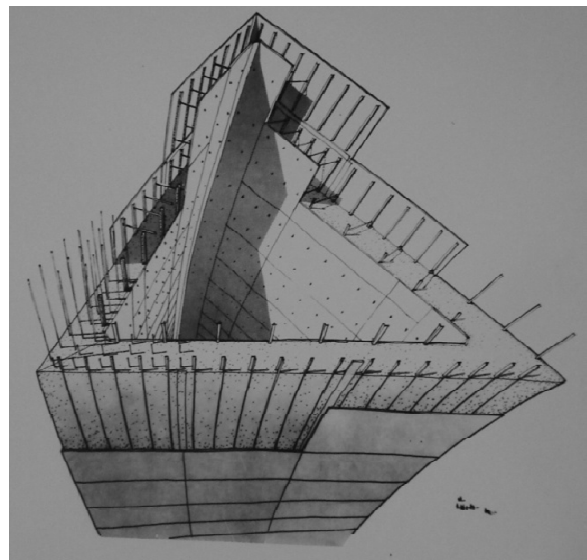


Figure 12. Equilateral cross-section of the Arch with inner and outer skin and post-tensioned concrete core. Courtesy of Yale University Archives, Eero Saarinen Collection.

²² In structural engineering design, *orthotropic* refers to a structure where an exposed steel plate surface is the primary structural element and is stiffened by perpendicular elements to improve its overall load-bearing capacity.

²³ *Post-tensioned concrete* is reinforced concrete in which a tensile force is applied to reinforcing strands after the concrete has cured.

²⁴ An-Di Nguyen, Jefferson National Expansion Memorial. *Spans*, July 2003, 1–3

Bandel was instrumental in determining the specific calculations for the weighted catenary form that were required for the Arch to be fabricated as designed. The angle of the curve, thickness of the legs, and relationship between the inner and outer skin were constantly changing. Bandel determined the mathematical formula by which the weighted shape could be calculated.²⁵

Because of the difficulty inherent in constructing an arched structure without centering, the legs of the Arch had to be designed to act as two cantilever structures.²⁶ Eventually, the legs would be joined at the top, upon which the overall strength of the Arch would be substantially increased. The design had to consider the loadings, stresses, and structural action at the various stages, while also addressing the practicalities of construction. Finally, since Arch was too tall for conventional cranes, the cantilevered legs had to be designed to support climbing cranes which would ride on rails attached to the outside face of the Arch legs.

Every element of the Memorial project was studied, drawn, and modeled. A full scale model of the grand staircase was erected on the lawn outside of Saarinen's office in Bloomfield Hills, Michigan. Since the slope of the staircase was derived from the weighted catenary shape of the Arch, the plywood mock-up was used to test the changing tread-riser relationship (Figure 13).



Figure 13. Saarinen at the top of the grand staircase mock-up outside his office in Bloomfield Hills, Michigan. Representatives of the NPS were in attendance. Courtesy of Yale University Archives, Eero Saarinen Collection.

Preparation for Arch Construction

George Hartzog, Jr., began work as the superintendent of JNEM on February 1, 1959. His forty-two month tenure was instrumental in developing the groundwork and making preparations for JNEM under a strict timeline and budgetary constraints. By the completion of Hartzog's appointment, the railroad relocation was approaching completion and a four-phase development program had been outlined for construction. Despite the limited funding, the project was kept on schedule through the scaling back of landscape and museum design components.

Saarinen believed the success of the Arch depended on its harmonious relationship with the adjacent setting. As interest in the project intensified, various publicly and privately funded developments were proposed in anticipation of the revived waterfront.

²⁵ Deborah Slaton and Mike Ford, interview with Bruce Detmers, April 1, 2009.

²⁶ *Centering* is a temporary framework for supporting an arch during construction until it is able to stand by itself.

Saarinen and the NPS realized the importance of monitoring developments beyond the limits of the site. For example, in 1959, Kansas City real estate developer Lewis Kitchen issued plans to construct a forty-two story building adjacent to the memorial. The structure would stand approximately 420 feet in height, nearly as tall as the proposed 590 foot height of the Arch.²⁷ Kitchen agreed to reduce the height of the proposed Mansion House on North Fourth Street. The negotiations with Kitchen reinforced the importance of controlling the surrounding development. In October 1959, St. Louis Mayor Raymond Tucker and NPS Director Conrad Wirth agreed to limit the height of buildings fronting the memorial site to 275 feet, approximately twenty-seven stories.

As part of the national effort to construct an interstate highway system and improve infrastructure, a steel-framed bridge was proposed at a location just south of the memorial park. The Missouri State Highway Department approached the NPS with plans for constructing an approach way to the Mark Twain expressway. The new roadway would be depressed below grade and be widened to encompass part of the memorial grounds. An agreement was made to allow the expressway to encroach on memorial property.

The NPS and the City of St. Louis imposed an ambitious construction schedule with the hope of completing the project in 1964, the bicentennial of the founding of the city. Phase I consisted of the relocation of the railroad tracks and construction of the tunnel. Phase II

involved the completion of exhibit research and planning, redevelopment of the levee, and excavations for the foundation of the Arch, museum, and visitor center. Phase III consisted of the construction of the Arch, museum, and visitor center. The project was scheduled to be complete in 1964 when Phase IV, final landscaping, was concluded. Saarinen considered the Arch to be the most important component of the memorial site, followed by the landscaping and then the museum. The priorities differed slightly for Hartzog, as was reflected in the phased construction schedule. Hartzog agreed that the Arch was the defining element of the project but required the museum to serve as an interpretive tool. The landscaping was a tertiary feature and the extent of its completion would be based on available funds. Thus, final site work was to be completed by the NPS as the final phase of construction.

JNEMA and the NPS quickly realized that the authorized funding would not be enough to complete the project on the proposed tight schedule. Additional funds were eventually provided, but not without eliminating design components to reduce the project cost. The size of the museum and visitor center was reduced to 41,500 square feet and was to include one finished theater, one roughed-out theater, a lobby space, restrooms, and limited office space.

From 1958, planning for the memorial project proceeded on three fronts. NPS personnel were occupied with research and development of museum exhibits; Richard Bowser was charged with creating the internal transportation system; and Eero Saarinen and his firm focused on the development of the Arch and visitor center.

The proposed memorial museum was to be the largest in the National Park system at the time

²⁷ Saarinen had not designed the Arch foundations and was considering changing the monument's height to 630 feet. Following discussion of nearby zoning, initiated by the planning of the Mansion House, Saarinen committed to a 630 foot Arch design. The Mansion House complex was completed in 1965 as a group of three twenty-eight story towers connected by a lower base.

and was to encompass a vast array of subjects and research materials. The museum exhibits were to capture the themes of westward expansion, as told through personal experience, and focused on the land, its acquisition, the pioneers, and the significance of expansion to the nation. Hartzog believed that construction of the museum should proceed simultaneously with that of the Arch. The museum was critical to understanding and interpreting the purpose for the establishment of the monument. The NPS Eastern Office of Design and Construction (EODC) disagreed with Hartzog and felt that the technical challenges associated with the Arch project demanded full attention. The NPS continued to devote its time to developing museum exhibits and every attempt was made to keep the museum as part of the initial construction phase. However, as the project continued to experience delays and federal funding became scarce, the completion of the interpretive museum was postponed until proper funding could be obtained.

As the design phase for the project reached completion, the project was met with tragedy. On September 1, 1961, Eero Saarinen died of a brain tumor in Ann Arbor, Michigan. Saarinen and Associates partners Joseph Lacy, John Dinkeloo, and Kevin Roche were entrusted with the task of completing the project.

The relocation of railroad tracks was the first phase of construction in the outlined four stage construction of JNEM. The railroad relocation project differed slightly from the Memorandum of Understanding issued in 1949. It consisted of the removal of the existing elevated track system and two of the three grade level tracks. One existing railroad track was to remain on the levee and a parallel track was to be reconstructed 100 feet inland from the elevated system. Two land bridges and a 960-foot-long ventilated tunnel were to

be constructed. On June 23, 1959, the long awaited ground-breaking ceremonies took place and demolition started on the Old Rock House.²⁸ The demolition and relocation of the elevated tracks followed in the spring of 1960. Construction of the tunnel and approach bridges and related levee redevelopment were finally completed in September 1963.



Figure 14. The new railroad tunnel nears completion to replace the old viaduct in the middle ground of this view, July 5, 1961. Source: JNEM archives, image V106-4205. Other photos in the JNEM archives document the tunnel construction process in detail.

²⁸ In early conceptual plans for the memorial, the restored Old Rock House was envisioned as a museum to the St. Louis fur trade. However, when plans were finalized in the 1950s, the original site of the Old Rock House lay in the path of the new railroad tunnel. The building was dismantled/demolished in 1959. There was some public expectation that the building would be reconstructed at the south end of the site, but no funding to reassemble or recreate the building was available. The original stone and timber materials were apparently discarded in the early 1960s. See Carolyn Hewes Toft, "The Arch Grounds and the Old Rock House," *Newsletter of the Landmarks Association of St. Louis, Inc.*, January/February 1996, <www.landmarks-stl.org>.

Construction of the Arch

Bids were accepted for the construction of the Gateway Arch and opened in a public ceremony held in the east courtroom of the Old Courthouse on January 22, 1962. The engineers of Eero Saarinen and Associates estimated that construction would take 875 days at a cost of \$8 million. The lowest bid was set by MacDonald Construction Company at \$11.9 million.²⁹

Due to the unexpected cost of construction, NPS Director Conrad Wirth announced that sufficient funds were not available to construct the proposed internal transportation mechanism, the innovative tram car system designed by Richard Bowser to transport visitors to the top of the Arch. The Bi-State Development Agency, established by the Missouri and Illinois legislatures to promote transportation in the St. Louis area, quickly provided a financial alternative in which Bi-State would issue bonds to provide funding for the installation of the tram system. Demand for the bonds was high and Bi-State was able to acquire over \$3 million, far above the estimated cost of nearly \$2 million.

Superintendent Hartzog resigned from the NPS on August 1, 1962, but remained indirectly connected to the project when he accepted a position as director of Downtown St. Louis, Inc. During his tenure, Hartzog oversaw the completion of the first phase of construction and was responsible for keeping the project on schedule and within the allotted funds by reducing the scope of the memorial and museum and making cutbacks. Upon his departure, the project was at a pivotal stage as Phase III construction began and physical progress on the memorial was seen for the first time in twenty-two years. Within a month, H.

Raymond Gregg took over the superintendency of the park.³⁰

H. Raymond Gregg served as superintendent of JNEM from 1962 to 1965, during the early construction of the Arch. His tenure was focused on balancing the progress of construction with the limited financing available. As the Arch was constructed, numerous delays were incurred resulting from labor disputes, refinement of construction practices, and questions of structural stability. Gregg retired from the NPS shortly before the Arch was completed.

Gregg's first months as superintendent were marked by concern for the budget. The expenditures of previous years' work had left very little remaining of the initial \$17.25 million authorized by Congress in 1958. Although the project could be sustained for the next few years through the already appropriated funds, the future completion of the project did not look favorable. Gregg had the difficult decision of either approaching Congress for additional funding, reducing the already bare scope of the project, or stopping the project due to lack of funding.³¹ After consultation with Director Wirth, the NPS reevaluated the cost estimates for completing the memorial and the decision was made to approach Congress and the City of St. Louis for an additional \$8 million in funding.³²

The United States Territorial Expansion Commission first approached Congress in November 1964 to explain the need for the additional funds. Hearings and proceedings worked their way through Congress during the remainder of Superintendent Gregg's tenure.

²⁹ Ibid., Chapter 7, 16.

³⁰ Ibid., Chapter 7, 19.

³¹ Ibid., Chapter 7, 21.

³² Ibid., Chapter 7, 20.

The Arch was designed as a six hundred and thirty foot weighted catenary arch with an orthotropic structure. The arch had two skins, an interior and exterior, which had slightly different curves that worked to structurally support each other. Additionally, the skins were connected through reinforcement bars and the cavity between them was filled with post-tensioned concrete up to a height of three hundred feet. The Arch was set on a concrete foundation and constructed of one hundred and forty-three prefabricated double-wall carbon steel and stainless steel segments.³³

Excavation for the Arch foundations required creating a pit for each leg at least seventy-five feet by ninety feet wide that extended to bedrock, approximately forty-four feet below grade. The vertical thrust of the catenary-shaped Arch relied on the stability of the bedrock and strength of the concrete foundation. Construction of the Arch began on June 27, 1962, when MacDonald Construction Company poured concrete for the south leg foundation. The foundation was constructed in five foot increments each of which demanded a continuous monolithic pour that took up to twenty-three hours and required 1,700 cubic yards of concrete.³⁴

³³ Throughout this document, the term *segment* is used to refer to the three-sided prefabricated elements that were placed one atop another to construct the Arch. The size of each segment decreases from the bottom of the Arch to the top. The term *station* is used to refer to specific locations on or within the Arch, numbered from Station 0 at the peak of the Arch to Station 71 at the base of each leg, as shown on the original construction drawings. Each station corresponds to the field weld installed to join adjacent segments during construction of the Arch. The numbering of stations and segments is such that Segment 63, for example, was placed on top of the gridline at Station 63.

³⁴ Don Haake, interview with Ted Rennison, March 24, 1981, 6–8.

As the foundations reached ten feet high, post-tensioning bars were installed. A second group of post-tensioning bars were started when the foundation reached twenty feet. In total, two hundred and fifty-two vertical post-tension bars were set into each foundation to stabilize the Arch during construction.³⁵

Work on the foundation and concrete shell of the museum was continued through the winter and required extensive protection and winter heating in order to ensure proper curing.³⁶ The Arch foundation was completed by February 1963 but work continued on the museum shell through the summer and into the fall. Meanwhile, the Pittsburgh-Des Moines Steel Company, a subcontractor of MacDonald Construction Company, was making preparations for the fabrication of the Arch segments.³⁷

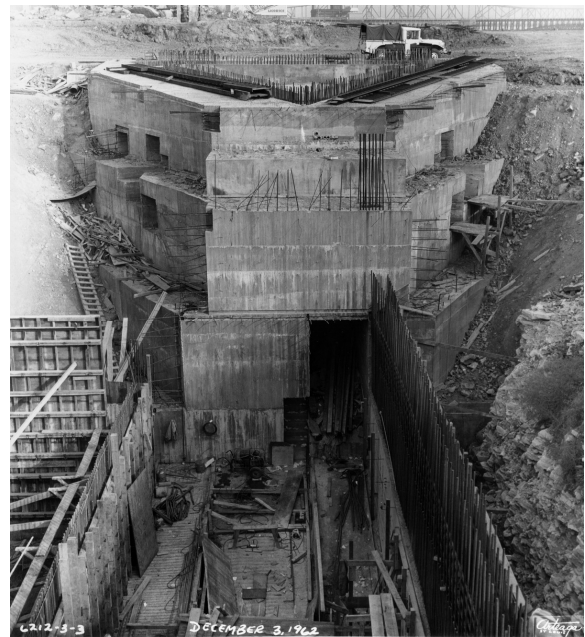


Figure 15. The north leg foundation nears completion, December 3, 1962. Source: JNEM archives, image V106-3938.

³⁵ Ibid.

³⁶ Ibid.

³⁷ Brown, Chapter 7, 18.

The beginning of the 1963 construction season presented a series of firsts, each marked by a ceremonial press event, in the long awaited construction of the Arch. The first segment was placed at the base of the south leg on February 12, 1963. Superintendent Gregg, along with members of the construction team, presided over a press conference to commemorate the occasion and answer questions.

Placement of the first steel sections introduced some minor difficulties to the project. The position of the foundation and post-tensioning bars were not in alignment with the angle of the steel segment. To rectify the situation, the post-tensioning bars were slightly adjusted and bent to fit within the segment. Hannskarl Bandel of Severud, Elstad, and Kruger recommended that additional reinforcing be added to compensate for the subsequent reduction in strength.³⁸

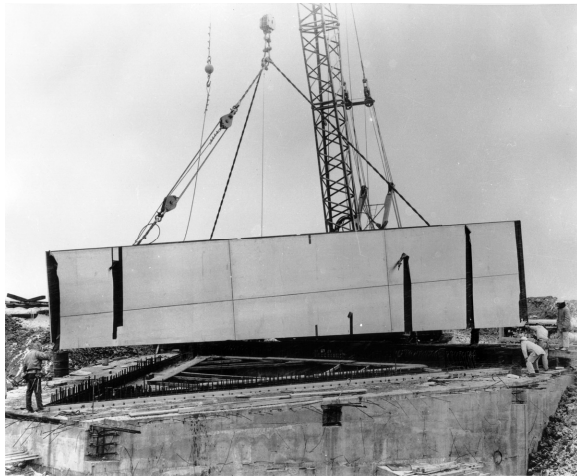


Figure 16. Placement of the first segment, February 12, 1963, at the south leg. Source: JNEM archives, image V106-3962.

On April 9, an informal ceremony was held as concrete was poured into the first above-ground segment. Symbolically, water from the Columbia River in Oregon was used in mixing the concrete to solidify the link between St. Louis and Fort Clatsop, Oregon, established by the Lewis and Clark expedition.³⁹

The contractors quickly established a systematic method and process of construction. The north and south legs of the Arch were erected simultaneously. Segments were assembled, hoisted into place, welded to the segment below, filled with concrete, and post-tensioned.⁴⁰

The Pittsburgh-Des Moines Steel Company was responsible for the fabrication and assembly of the steel segments. The first four segments were all entirely shop assembled as one large triangular segment then shipped to the site and erected into place. The remaining segments were partially fabricated in the shop and welded together in their final configuration on site. The larger segments, at the bottom half of the Arch above the first four, were fabricated as three double-wall flat panels and were assembled on site by installing a continuous vertical weld at each of the three corners. Pick points were welded at the inside intrados corners to accommodate the creeper crane lift cables.⁴¹ All segments above the three hundred foot level were fabricated as three L-shaped pieces. Field welds for the upper segments were made on the faces of the panels, not at the corners.⁴²

³⁹ Brown, Chapter 7, 21.

⁴⁰ Ibid., Chapter 7, 21–22

⁴¹ *Intrados* is the interior curved surface of an arch. Similarly, the *extrados* is the exterior curved surface of an arch. A *pick point* is an attachment point on a structural element designed to support moving and placement of the element during construction.

⁴² Ken Kolkmeier, interview by Dan Worth of BVH; Stephen Kelley of WJE; Robert Moore, NPS JNEM

³⁸ Rennison, 8.

Once the pieces were fabricated and assembled, they were carefully lifted into place using cranes. When the Arch legs reached 60 feet in height, creeper derricks were erected to facilitate the construction as further discussed below.

After the steel segments were hoisted into place and aligned, both the interior and exterior skins were tack welded into place and allowed to sit overnight while the survey team verified the height and location.⁴³ In the following days, the segment was secured with a continuous weld. Because welding was done on a vertical surface with access from only one side, numerous weld passes and grindings were required to guarantee a complete, 100 percent, weld. A backup bar was installed on the back side of the steel prior to setting each segment to assist in the effectiveness of the welding process. Random samples of the welds were X-rayed to verify quality. The process was labor intensive and demanded skilled welders who would work in the extreme heat and confined environment.⁴⁴ Welders with experience at Titan II missile construction sites were brought in to work on the Arch.⁴⁵

Once the concrete had adequately cured to 4,000 psi strength, the post-tensioning bars were torqued up to seventy-one tons each.⁴⁶ The process of post-tensioning made the concrete more effective at handling tensile stresses applied by the partially completed Arch legs. The post-tensioning bars were connected by a threaded sleeve and encased in a hollow steel sleeve to allow for uniform elongation. Any unforeseen bends,

inconsistencies in integrity, or non-uniform torque of the bars could greatly affect its tensile strength.⁴⁷

With only a few segments in place, the project experienced its first construction delay. The Hoisting and Operation Engineers in St. Louis went on strike on May 1, 1963, and progress on the Arch project was brought to a halt. The strike lasted twenty-six days and forced a change in the construction schedule. The goal of finishing the Arch in time for the city's bicentennial seemed out of reach as a revised completion date of February 1, 1965, was set.

Delays to the Arch construction translated into delays in the museum construction. The museum shell was scheduled for completion on October 16, 1963. However, a final inspection revealed that substantial work was still required before the project would be complete. MacDonald Construction Company was assessed damages for every day the completion of the project was delayed. The NPS finally accepted completion of the museum portion of the MacDonald Construction contract on January 13, 1964.⁴⁸

The interior staircase of the Arch was designed by Eero Saarinen and Associates and fabricated by Southwest Ornamental Iron in Kansas City. The framework arrived in June 1963 and installation proceeded the following month. The assembly of the stairs kept pace with the construction of the exterior of the Arch.⁴⁹

Historian; Al O'Bright, NPS Historical Architect; and Victoria Dugan of NPS JNEM, January 14, 2009.

⁴³ Ibid., 4–5.

⁴⁴ Rennison, 9–10.

⁴⁵ Kolkmeier, interview by Worth et al., January 14, 2009.

⁴⁶ Ibid.; Rennison, 8.

⁴⁷ Rennison, 16.

⁴⁸ Brown, Chapter 7, 23–24.

⁴⁹ Ibid.

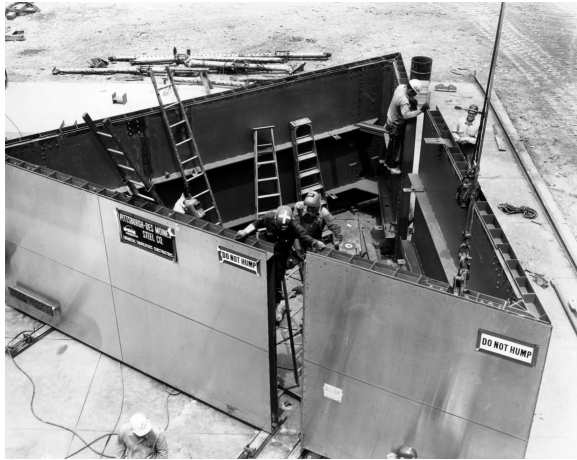


Figure 17. Assembly of an Arch segment from prefabricated L-shaped pieces, with field welding on the face of each side. This segment was installed above the 300 foot level. Photograph courtesy of Ken Kolkmeier.



Figure 18. The Arch legs under construction with the creeper cranes in place, late summer 1963. Source: JNEM archives, image V106-4034.

In July 1963, when the Arch reached 60 feet in height, creeper cranes were built to complete the construction process. The creeper cranes were designed by Richard Gardens and fabricated by the Pittsburgh-Des Moines Steel Company. Each leg of the Arch had its own crane that was used for hoisting the steel segments and putting them in place. Dual tracks were constructed along the face of each Arch leg, and platforms were assembled to support the cranes. As the creeper derricks proceeded up the Arch, the back legs could be adjusted so that the work platforms remained level.⁵⁰ The cranes were controlled from operator's cabins on the ground. The operators often could not see what they were doing and relied on telephone communication from the derricks to navigate the cranes.⁵¹

Post-tension Failures

When construction reached Station 62 of the south leg in the autumn of 1963, issues arose over post-tensioning.⁵² At this point in the construction, the post-tensioning bars were up to 120 feet in length and were calculated to have an elongation of almost 6 inches. However, the elongation of the bar was measured at 3 inches, half of the expected amount. Construction on the south leg was stopped by Eero Saarinen and Associates and their structural consultants Severud, Elstad, and Kruger until the cause of this problem could be isolated and addressed.

A piece was cut out of the stainless steel skin of Segment 62 of the south leg, which had been placed on September 16. (The exact location, size, and later repair methodology used at this swatch could not be determined from the available reference material as part of

⁵⁰ Kolkmeier, interview by Worth et al., January 14, 2009.

⁵¹ Bill Quigley, interview with Robert Moore, October 28, 1995.

⁵² Brown, Chapter 7, 23

this study.) Upon excavation of the concrete core, it was realized that the threaded couplers connecting the post-tensioning bars were restricting the elongation.⁵³ The coupler was replaced, clamped into place, and the sleeve was infilled with grout to prevent future corrosion of the bars. The steel piece was replaced and concrete was pumped in to refill the voids between the interior and exterior skin at this location.

MacDonald Construction Company issued a plan for correcting this type of deficiency on November 18, 1963. The couplers and overstressed post-tensioning bars were removed, new couplers and bars installed, and the opened cavity was filled with a concrete and fly-ash grout material to prevent future corrosion of the bars. Additional reinforcing bars were added to the structure to compensate for any damage done during the repair process. The failure of the bars and subsequent work stoppage resulted in a one and one-half month delay in construction.⁵⁴ A formal change order accepting the proposed correction was signed by Superintendent Gregg in December.⁵⁵ Segment 61 on the south leg was placed on January 10, 1964. The issue of post-tensioning put the south leg 48 feet behind the pace of the north leg before all Arch construction was halted in 1964 by concerns over structural stability at Station 45.



Figure 19. Applying tensile stress to the post-tensioning bars, 1963. Source: JNEM archives, image V106-3974.



Figure 20. The Gateway Arch as seen during Independence Day ceremonies in July 1964. Courtesy of Yale University Archives, Eero Saarinen Collection.

⁵³ From available information, it is not clear whether a single coupler was changed out or whether others were also replaced. According to Ken Kolkmeier in discussion of post-tensioning rod problems, repairs may have taken place in a number of locations. (Kolkmeier, interview with Worth et. al.).

⁵⁴ Rennison, 16–17.

⁵⁵ Brown, Chapter 7, 23, 25.

Station 45 Structural Stability

At Station 45, approximately 300 feet high, plans called for a change in the structural assembly of the Arch. Below Station 45, the cavity between the interior and exterior steel skin was filled with concrete and post-tensioned steel. At this point in the construction process, the legs were designed to lean 49 feet towards the center. According to Severud, Elstad, and Kruger, the reinforcing system allowed the unrestrained legs of the Arch to remain stable during erection. The Arch was designed so that structural reinforcing established in the first segments of the structure could maintain the stress and strain yet provide the flexibility required for the remainder of the construction process.

On June 23, 1964, the NPS issued a stop order on the fabrication of the segments. Construction was not to continue beyond Station 45, the top of the concrete fill. Richardson-Jordan, the structural engineering consultant to the Pittsburgh-Des Moines Steel Company, issued a formal complaint about the feasibility of the Arch structure. They believed that the unsupported segments would buckle when the Arch legs were pulled apart during installation of the final piece. Their doubts were supported by the Bureau of Reclamation of the Department of the Interior, which was responsible for the design and review of bridge projects. The agency performed a structural study on the Arch and came to the same conclusion as Richardson-Jordan. Additional studies were conducted by the Bureau of Public Roads to test seismographic measurements and determine the anticipated movement and sway of the Arch.

Joe Jensen, Assistant Director of the NPS, met with representatives of Eero Saarinen and Associates and Severud, Elstad, and Kruger to discuss the stability of the Arch. The design team presented their position that the tests were based on faulty assumptions and

inadequate information. Research relating to the properties of stainless steel, thermal flow characteristics of the dual skin system, testing of a prototype three-panel system, and wind tunnel tests had not been incorporated in the Bureau of Reclamation's report. It was agreed by the Bureau that further research was required before a final verdict could be determined. Until that time, work could commence. On July 2, 1964, fabrication of the segments was resumed.⁵⁶

At Station 45, the concrete infill and post-tensioning bars were supposed to terminate. To secure the top of the concrete cavity and counter balance the tensile stress introduced by the vertical post-tensioning bars, hundreds of horizontal post-tensioning bars were installed within the wall cavity and tightened to put compression on the vertical bars.⁵⁷

Above the concrete-filled cavity, the interior and exterior skins were connected by "L" brackets with the short leg spot welded to the inner skin and the long leg securing the outer skin. Upon installation of Segment 45 on the north leg on September 27, 1964, it was noted that ripples in the stainless steel skin occurred every two feet in accordance with the locations of the diaphragm brackets. The rippled appearance may have been more noticeable than anticipated as the stainless steel skin was thinner than the original design assumed.

This segment was removed on October 30, 1964, and various attempts were made to resolve the warping. Under the direction of Hannskarl Bandel, who was concerned that the various repair attempts may have compromised the structural integrity of the segment, Segment 45 was reinstalled on November 17, 1964, and the wall cavity filled

⁵⁶ Brown, Chapter 7, 25–26.

⁵⁷ Rennison, 18.

with a lightweight concrete in an effort to stabilize the segment. Segment forty-five on the south leg was also filled with concrete to match the north leg.⁵⁸ Subsequent segments were installed with L-brackets as intended by the original design, and the associated ripples were accepted. (Refer to the Structural description, below, for further information on the installation system.)

CORE

As the site of the Dred Scott and Virginia Minor trials, St. Louis had a history of influential civil rights cases which was not forgotten during the construction of the Arch. In May 1964, the St. Louis chapter of the Congress of Racial Equality (CORE) alleged that contractors and subcontractors on the Arch project followed discriminatory practices. Research conducted by CORE revealed that no African-American workers were employed in the skilled trades working on the memorial project. Furthermore, the Building and Construction Trades Council, the St. Louis chapter of the AFL-CIO, had never admitted any African Americans or supported any African-American workers through the apprenticeship training program.⁵⁹ CORE organized protests and picket lines in front of the Old Courthouse to publicize the exclusion of black workers.

On July 14, 1964, Percy Green and Richard Daly, members of the Action Committee to Improve Opportunities for Negroes (ACTION), made their way onto the construction site, climbed 120 feet up a construction ladder, and positioned themselves on the Arch structure. When workers returned from lunch, they were unable to get around the protestors and resume work. The stalemate lasted for four hours, as the activists voiced their demand for a ten percent African-

American workforce to the onlookers below, until Assistant Superintendent LeRoy Brown went up to the protestors to listen to their demands and persuade them to descend. The police then arrested them.⁶⁰ The controversy brought national interest to the civil rights movement and the issue of enforcing the Federal Equal Employment Opportunity Program.

Completing the Arch

As the Arch approached 530 feet in height (approximately Station 22), work proceeded to install the stabilizing strut designed to prevent excessive leaning. The legs were leaning 150 feet inwards, and together with the extra weight of the creeper cranes, additional support against overturning was required as part of the design. The Pittsburgh-Des Moines Steel Company had proposed the construction of a high-strength, light-weight stabilizing strut that would bridge between the two legs, allowing them to stabilize each other. The 225-foot-long, bridge-like stabilizing strut structure was assembled on the ground and hoisted into place on the morning of June 17, 1965; this effort became a media event. The stabilizing strut forced the legs of the Arch to be jacked outward 6 feet.⁶¹ The Pittsburgh Des-Moines Steel Company had placed large letters on the strut and creeper derricks (two erection tools they were credited with designing) with the firm's initials, "PDM". This advertising was not well received by the NPS, which ordered the signage removed. It was not until two months later, when acting Superintendent LeRoy Brown deducted \$225,000 from payment to the MacDonald Construction, that some of the large letters

⁵⁸ Ibid., 21.

⁵⁹ Ibid., Chapter 7, 25.

⁶⁰ Robert J. Moore, Jr., *Showdown under the Arch, the Construction Trades and the First "Pattern or Practice" Equal Employment Opportunity Suit, 1966*, Gateway Heritage (St. Louis, Missouri: Missouri Historical Society, Winter 1994–1995), 34.

⁶¹ Rennison, 21–24.

were finally removed.⁶² The entire group of letters was not removed until the stabilizing strut came down.

LeRoy Brown officially assumed responsibility of superintendency of JNEM on August 1, 1965, following the retirement of Raymond Gregg. Brown had previously served as JNEM Assistant Superintendent and had overseen most aspects of Arch construction for over a year. At the time of his superintendency, the Arch structure was nearing final closure but there was still debate regarding its structural stability. After the superstructure of the Arch was complete, it would be Brown's job to facilitate the installation of the interior Arch systems, construction of the museum and visitor center, and development of the landscaping in a timely manner with the limited funds remaining. The task proved more daunting as the number of contractors and complications during construction increased.

Throughout the construction of the Arch, there was concern that the two legs would not meet at the center. Minute discrepancies in weld thickness or placement of the steel segments could dramatically affect the structural stability and final installation of the keystone segment. To monitor the progress, nightly measurements were made, using a theodolite and geometric calculation, to verify the consistency of construction.⁶³ Throughout the construction process, the difference between the heights in the two legs, as taken at night, remained less than one inch.



Figure 21. View of Arch construction and the stabilization strut. Courtesy of Bruce Detmers.



Figure 22. The Arch nears completion with the stabilizing strut in place, September 9, 1965. Source: JNEM archives, image V106-4124.

⁶² Brown, Chapter 7, 29.

⁶³ A *theodolite* is a surveying instrument that uses a telescopic site to establish precise horizontal or vertical angles.

Discrepancies between the height of the north and south legs were observed during the daylight hours. Throughout the day, the heat of the sun, shining more directly on the south leg, caused that leg to elongate and deflect downward 14 inches below the level of the north leg. For this reason, the Arch project team requested that the final piece be installed at night, when temperatures were consistent and the height of the legs was even.⁶⁴

However, this approach was rejected by the City of St. Louis, and the installation of the final segment was performed during daylight hours so that a public ceremony could be held at the completion of the Arch structure.

On October 28, 1965, the Arch hosted a “topping out” ceremony when the final 8-foot-wide segment was inserted into the Arch. The media event was attended by Undersecretary of the Interior John Carver and presided over by Superintendent LeRoy Brown. Members and supporters of the project team, as well as those who had expressed doubt of the structural stability, politicians, the media, and hundreds of onlookers waited in anticipation for the events of the day. For Eero Saarinen and Associates; Severud, Elstad, and Kruger; and the NPS, the topping out was the culmination of a vision that had been decades in the making and a validation of their faith in the controversial and innovative structural design.

The ceremony was scheduled for the morning before the south leg was heated by the sun. The local fire department sprayed the leg with cold water to keep the Arch cool. With the application of 500 tons of pressure using hydraulic jacks between the creeper cranes, the topmost segments of the north and south legs were pried 6 feet apart. The final piece had been temporarily retrofitted with five inch pins to help secure a quick fit with the north

and south segments. As the segment was lowered, the pins were inserted into the north segment and pushed into place. The south segment was raised approximately five inches until it was aligned with the keystone piece, and as the five hundred ton pressure was relieved, the gap between the south leg and center segment disappeared. The legs lined up perfectly.⁶⁵

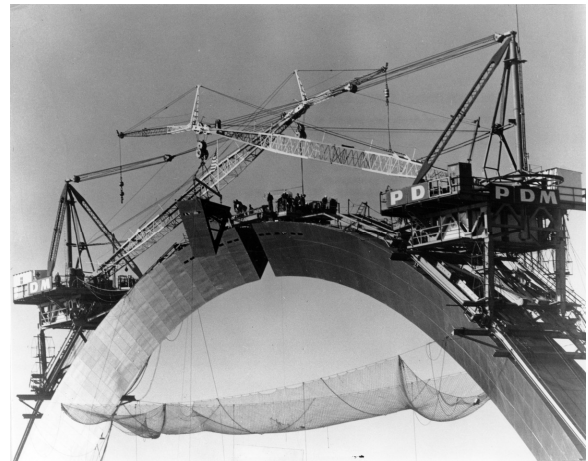


Figure 23. Installing the final segment of the Arch, October 28, 1965, at the top of the north leg. Source: JNEM archives, image V106-4131.

After the keystone segment of the Arch was inserted, final cleaning, repair, and polishing could begin. The stainless steel panels were washed and polished by hand. Bolt holes in the exterior skin were plugged with stainless steel punches salvaged from the Pittsburgh-Des Moines Steel Company manufacturing plant during fabrication. The stainless steel plugs were welded and ground smooth. The cleaning created some inconsistencies in the finish. Hand polishing never seemed to produce the same result as the shop finish, and patched areas remained visible to the discerning eye. The locations of the stabilization struts required extensive cleaning and polishing in order to have an aesthetically pleasing appearance. The winter weather

⁶⁴ Rennison, 25.

⁶⁵ Ibid., 25–26.

complicated the cleaning process, as water-based products turned to ice. Final preparations of the stainless steel skin continued as the creeper derricks inched their way down the Arch.⁶⁶ In the fall of 1966, the derricks were disassembled and the cleaning of the Arch was complete.

Once the Arch structure was completed, focus was placed on construction of the visitor center, mechanical systems, tram, and landscaping. By October 1965, limited progress had been made on all three components as the subcontractors were forced to work around the schedule of the structural construction.



Figure 24. Cleaning the Arch and removing the creeper cranes, February 7, 1966. Source: JNEM archives, image V106-4147.



Figure 25. Aerial view of the top of the Arch during cleaning. Courtesy of Yale University Archives, Eero Saarinen Collection.

Visitor Center and Arch Interior Construction

In October 1965, after a year of soliciting funds, Congress authorized the appropriation of an additional \$6 million for completion of the visitor center, museum, mechanical systems, and landscaping, bringing the total budget to \$23,250,000 since 1935. In order to acquire the necessary matching funds from the City of St. Louis, a bond issue would be required. However, despite the excitement generated by the recent completion of the Arch, the 1966 bond issue failed to capture the two-thirds popular vote required.⁶⁷

In an attempt to generate support for a 1967 bond issue election, the NPS held an open house on the mud filled construction site to better express the need for the city's matching funds. The bond issue passed in a second election on March 7, 1967.⁶⁸

The tight budget for the memorial was most distinctly felt in the completion of the visitor center and museum. As the Arch structure neared closure, the NPS was prepared to begin work on the Arch interior and visitor center.

⁶⁶ Rennison, 38.

⁶⁷ Brown, Chapter 7, 27.

⁶⁸ Ibid., Chapter 7, 36, 39.

Despite all bids being over the proposed budget, in November 1965 the NPS offered the contract to Hoel-Steffen Construction Company, the lowest bidder. The estimated completion date for the project was July 1, 1966.

Part of the acceptance of Hoel-Steffen as contractor for the museum construction depended on their compliance with equal opportunity requirements. The publicity generated by CORE during the Arch construction had made the NPS acutely aware of their national role in enforcing the Federal Equal Employment Opportunity program. As a result, Hoel-Steffen agreed to provide more construction jobs for African Americans. No local minority plumbers were members of the AFL-CIO. In an effort to comply with equal opportunity requirements, the company subcontracted with the African-American owned E. Smith Plumbing Company, associated with an alternative union, the Congress of Independent Unions. On January 7, 1966, the Building and Construction Trades Council, an AFL-CIO affiliated union in St. Louis, unanimously voted to stop work on the project, referencing their policy that union members work only on projects where all workers were AFL-CIO members.

Two weeks later, the Department of Labor took action against the labor union, citing President Johnson's 1965 Executive Order on equal opportunity. On February 5, 1966, the Justice Department filed their first ever "pattern or practice" suit against the AFL-CIO and four member unions on charges of discrimination, specifying that the unions did not offer equal opportunity for African-Americans in the building trades.⁶⁹ The suit

was followed by a temporary injunction that required unions to return to work effective February 9. Labor disputes caused a fifty-one day delay in the construction of the visitor center.

The labor dispute and resulting litigation had the direct effect of African-American workers being hired as journeymen and apprentices in the St. Louis steamfitters and plumbing unions. The Justice Department lawsuit was eventually heard at the United States Court of Appeals for the Eighth Circuit, which agreed with the government that St. Louis unions engaged in discriminatory hiring practices.⁷⁰

The Arch air-conditioning system was installed by Hoel-Steffen Contractors as a two-duct system. Since the ventilation system had been custom fabricated to fit the preconceived spaces of the visitor center and unique area created by the Arch structure, the ductwork was assembled on site and welded in place. The laborious process contributed to the extensive delays to the completion of the visitor center, internal transportation system, and final dedication.

Throughout the construction process, it was apparent that the interior environment of the steel Arch would be difficult to regulate. High humidity, heavy condensation, and varied daily temperature swings were characteristic of the space. This type of atmosphere was not conducive to proper performance and maintenance of the mechanical and electrical systems needed to sustain the visitor center and internal transportation network. The project was plagued by corroded mechanical equipment and deteriorating electrical systems subjected to the Arch microenvironment.

⁶⁹ Robert J. Moore, Jr., *Showdown under the Arch, the Construction Trades and the First "Pattern or Practice" Equal Employment Opportunity Suit, 1966*, Gateway Heritage (St. Louis, Missouri: Missouri Historical Society, Winter 1994–1995), 30.

⁷⁰ *Ibid.*, 40–41.

Elevator Tram Installation

Richard Bowser was a second generation elevator man. Having dropped out of the University of Maryland to join the Navy, Bowser returned from World War II and began working for his father. The father and son team developed, manufactured, and installed Bowser Parking System elevator equipment which did not utilize ramps. The Bowser Garages were unique in that the elevator system could travel horizontally and diagonally as well as vertically. Thirty-five Bowser Garages were built.

Saarinén hired Bowser as an independent engineer to design the internal transportation system that would bring visitors to an observation level at the top of the Arch.⁷¹ His unique design consisted of eight capsule cars, each holding five people, connected to an elevator motor and cable system in each leg of the Arch. As the tram was hoisted up the Arch, each capsule rotated within its own ring-shaped frame. The weight of the passengers helped to keep the capsule upright. Each leg of the Arch contained its own independent transportation system.⁷²

As noted above, original financing for the construction of the tram was obtained through bonds issued by the Bi-State Development Agency as arranged in an agreement between Bi-State and the NPS in 1962.⁷³ Upon

completion of the transportation system, Bi-State would provide day-to-day operation of the system. The task of installing the unique apparatus was given to MacDonald Construction Company as part of the completion of the exterior Arch construction contract. The NPS required that the tram system be fully operational within 125 days after the Arch was completed. Unfortunately, construction of the tram encountered numerous challenges and setbacks, which forced delays in the completion of the system.⁷⁴

In March 1965, responsibility for fabricating the sixteen passenger capsules (eight for each leg of the Arch) was contracted to General Steel Industries, Inc., St. Louis Car Division. Within two months, the installation of the foundation and electrical system for the elevator and train were proceeding.⁷⁵ As construction commenced, delays were incurred due to difficulties with coordination of contractor schedules, moisture and ventilation issues in the interior of the Arch, and safety testing of the tram system.

Despite the many delays, the north leg tram was officially opened to the public with a ribbon cutting ceremony on July 24, 1967. Bi-State was then able to start generating funds to finance its interest accruing bonds. Completion of the south leg tram came eight months later. However, it was not until May 18, 1968, that both trams ran simultaneously.⁷⁶

⁷¹ Robert Moore, *Dick Bowser and the Arch's Unique Tram System*, JNEM website accessed October 23, 2008. <http://www.nps.gov/jeff/planyourvisit/dick-bowser.htm>.

⁷² Robert J. Moore, Jr., "Give Us a Concept in Two Weeks: Dick Bowser and the Arch's Unique Tram System," *Museum Gazette* (St. Louis: JNEM, July 1993).

⁷³ The Bi-State Development Agency was created in 1949 through an interstate compact between the states of Missouri and Illinois, with responsibility for public transportation in the St. Louis metropolitan region. The agency operated the St. Louis streetcars

until their discontinuation in May 1966 as well as the city buses. Today, the agency owns or operates St. Louis Downtown Airport, the adjoining industrial business park, paddlewheel-style river excursion boats, the MetroLink light rail and bus system, and the Arch parking garage, as well as the Arch tram system.

⁷⁴ Brown, Chapter 7, 22.

⁷⁵ *Ibid.*, Chapter 7, 28.

⁷⁶ *Ibid.*, Chapter 7, 45.



Figure 26. Installing wall cladding inside the observation level, May 5, 1966. Source: JNEM archives, image V106-4158.



Figure 27. Painting the interior of the Arch legs, August 4, 1966. Source: JNEM archives, image V106-4167.

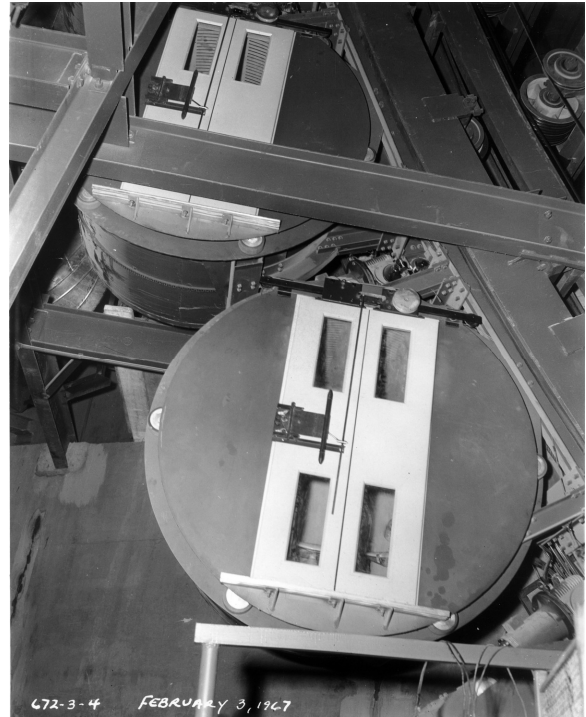


Figure 28. Installing the tram system, February 3, 1967. Source: JNEM archives, image V106-4192.

Landscaping

Landscape architect Dan Kiley, working as an independent consultant, prepared plans for the completion of the memorial site, which were accepted by the NPS in 1966. Kiley had been part of Saarinen's original competition design team. Unfortunately, in 1966 the NPS did not have funding available for the landscape work and the drawings would serve only as a guide for landscaping the grounds in the future. Instead of Kiley's plan for broad sweeping walkways, dense forests edged with flowering trees, and two large meadows with lagoons, the site remained bare with grass and dirt.

In 1969, the NPS generated a low budget plan based on Kiley's design. Following the dedication of the Arch, criticism had been directed toward the landscaping of the site. Initial efforts had focused on creating a plan that would address the needs of the site immediately surrounding the Arch. In the

design effort initiated in 1969, attention was given to the installation of waterproofing on the visitor center roof, addressing issues of drainage and grading, and improving the appearance of the north-south axis defined by the Arch. The documents were completed in August 1969.⁷⁷



Figure 29. Completing the north entrance ramp to the visitor center, October 6, 1966. Source: JNEM archives, image V106-4175.

Dedication

By 1968, the Arch, internal transportation system, and visitor center were complete. The NPS had exceeded spending limits provided by the federal government and the bonds issued by the city of St. Louis. Completion of the museum, restoration of the Old Courthouse, and final landscaping would have to wait until new funds could be appropriated.

The day for the final dedication of the Arch was set for May 25, 1968, the twentieth anniversary of the United States Territorial Expansion Memorial Commission's acceptance of Eero Saarinen's original design for the Gateway Arch. A two-day celebration was planned to dedicate the memorial,

commemorate the vision of westward expansion, and honor the determination and persistence of those who tirelessly contributed to the creation of JNEM.

When the day arrived, the guests were greeted with an unusually heavy downpour. Water drenched the site, washed down the entrance ramps and flooded the visitor center. There were no alternate plans in case of inclement weather and many of the festivities would be cancelled. Inside the visitor center, the dedication ceremony continued and culminated with Vice President Hubert H. Humphrey's address.⁷⁸ Although weather precluded the glorious event envisioned by Mayor Alfonso Cervantes and Secretary of the Interior Stewart Udall, the ceremony was a symbol of the hard work, sacrifice, and passion from which the memorial project had grown.



Figure 30. Visitors at the observation level, circa 1968. Source: JNEM Photo Reference Collection, image VPRI-1379.

⁷⁷ Regina M. Bellavia and Gregg Bleam, *Cultural Landscape Report* (Omaha, Nebraska: NPS, 1996), 86.

⁷⁸ Brown, Chapter 7, 46–47.

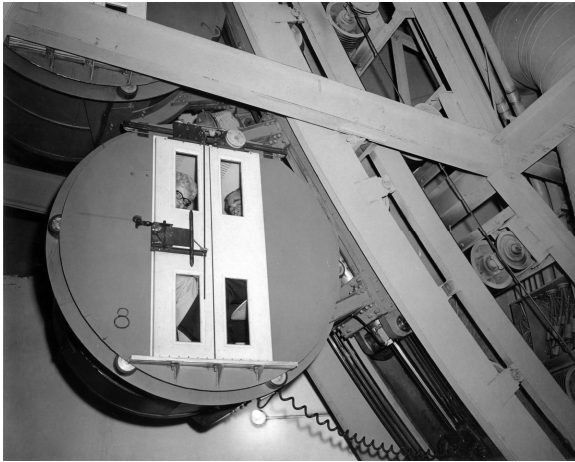


Figure 31. Visitors ascend via the completed tram, circa 1968. Source: JNEM archives, image VPRI-1392.

Maintaining the Arch, 1968–2009

Upon completion of the Arch, attention was focused on completing the Museum of Westward Expansion and landscape. Shortly after the official dedication, Superintendent LeRoy Brown was promoted, transferred elsewhere, and replaced by Dr. Harry W. Pfanz. The new superintendent was successful in securing federal funds to support the extension of the visitor center/museum complex. A small number of exhibits were opened by 1971 and an extension to the main lobby, an additional theater, administrative areas, and concessionaire space were completed during the following year.⁷⁹

Simultaneously, progress was made in initiating the first phase of landscaping. In 1972, Suburban Tree Service of St. Louis was contracted to plant the first trees outlined in Dan Kiley's 1966 landscape plan.

In 1973, as federal and city funds dwindled, the NPS agreed to focus its efforts on completion of the Museum of Westward Expansion by the United States bicentennial. Plans were generated for interpretive exhibits

and construction of the grand staircase, a stairway linking the Arch to the riverfront.⁸⁰ In August of 1976, construction was completed and the museum was officially opened.

With the bicentennial and opening of the museum and grand staircase came an increase in publicity and attendance. Additional federal funding was appropriated in 1976 and focus was directed toward completing the landscape, the last component of Saarinen's design. The final phase of landscape improvements was implemented in 1980 and completed by the fall of 1981.⁸¹

As the museum and landscape components of the JNEM were being completed, minimal maintenance was required for the stainless steel arch and transportation system. However, in the 1980s, continuous use and flooding warranted repair and replacement of materials within the Arch and visitor center.

By 1983, the exterior terrazzo ramps providing access to the visitor center were deteriorating. Under the guidance of WVP Corporation and RL Praprotnik, the terrazzo was replaced with granite pavers over new waterproofing and an electrical ice-melt system. The work was completed in 1985.⁸²

Major flooding in 1987 and water infiltration observed through the visitor center roof resulted in the need for a study of water runoff. During the investigation by Zurheide-

⁸⁰ The staircase was not built according to Saarinen's specifications. The stairs were constructed not of granite but rather concrete, with a normal tread-riser relationship. Also, only the outer edges of the staircase were built in 1976, while the middle zone was constructed in 2001.

⁸¹ Ibid.

⁸² Robert Moore, *Urban Innovation in Practical Partnerships: An Administrative History of Jefferson National Expansion Memorial, 1980–1991* (Washington, D.C.: GPO, 1994)

⁷⁹ Brown, Chapter 8

Hermann, Inc., several cracks were discovered in the ceiling support beams of the visitor center. Structural concerns in conjunction with the roof leaks led to removal of earth above the visitor center. A reinforcing mesh and waterproofing were installed to improve water drainage and increase load capacity. The earth was replaced with lightweight concrete. In addition, interior repairs were made to address cracking in the beams. Steel angles were bolted around damaged beams to provide additional support. The waterproofing and structural repairs were completed in 1990.⁸³

In 1989, the Arch tram ticket area, run by Bi-State Development Agency, was remodeled. The renovation, designed by the St. Louis-based firm of Hellmuth, Obata, & Kassabaum (HOK), called for the installation of a supplemental HVAC system, reconfiguration of the ticket queue, and removal of carpeting with subsequent restoration of the original terrazzo flooring beneath. Original carpeting in the Museum of Westward Expansion was removed in 1989 and replaced the following year by a carpet donated by Allied Fibers.⁸⁴

After twenty-one years of service, heavy pumps used to keep the load zones free of water and flooding were replaced in 1989. The pumps were located approximately forty-five feet below grade, near the lower tram load zone. In order to facilitate removal of the original pumps, an I-beam superstructure was constructed within the Arch/visitor center to hoist the machinery.⁸⁵

1993 Flood

Situated along the Mississippi River, the JNEM was subject to numerous floods. High water levels in 1973, 1981, and 1987 all resulted in the need for repairs, alterations,

and a comprehensive clean-up of the site. In 1993, the Midwest was affected by hundred-year floods. Abnormally high rainfalls led to high water levels and rapid currents. The turbulent waters undermined levees along the Mississippi and tributary rivers and compounded the flooding.⁸⁶ At JNEM, waters reached halfway up the grand staircase (Figure 32). The record high flood waters seeped into the subsurface spaces of the Arch and visitor center structure. Water infiltration resulted in localized damage and necessitated clean-up of interior service spaces and tram load zone areas. Electrical and sewage systems were most significantly affected.



Figure 32. The Gateway Arch and the Mississippi River at the peak of the flood, August 1, 1993. Source: Missouri Highway and Transportation Department.

Operations Maintenance Technology

Improvements in technology were implemented into the Arch operations, maintenance, and management plan to increase efficiency. In 1985, an Energy Management System (EMS) was installed by Mack Electrical Company. The system allowed for automatic control of the chiller

⁸³ Ibid.

⁸⁴ Ibid.

⁸⁵ Ibid.

⁸⁶ R.E. Southard and B.J. Smith, *Flood of 1993—Mississippi River Near the Jefferson National Expansion Memorial (Arch), St. Louis, Missouri* (Washington, D.C: U.S. Department of the Interior, U.S. Geological Survey, October 1995).

and air-handlers. By 1986, the automated system was expanded to include the entire HVAC system. The purpose of the upgrade was to reduce the need for constant monitoring of the interior environment and to reduce costs and increase the energy efficiency of the system. Unfortunately, the projected savings associated with EMS were not realized at the Arch.⁸⁷

The JNEM and Indiana Dunes National Lakeshore along the shores of Lake Michigan became the subject of pilot studies for the NPS Midwest Region in implementing the Maintenance Management System (MMS). Started in 1987, the computerized information database was designed to record and establish a cyclical maintenance plan for the park. By the 1990s, the computer program had been expanded to document landscape, transportation, and general maintenance programs for the park.⁸⁸

⁸⁷ Moore, *Urban Innovation in Practical Partnerships: An Administrative History of Jefferson National Expansion Memorial, 1980–1991*.

⁸⁸ Ibid.

CHRONOLOGY OF DEVELOPMENT AND USE

The construction history for JNEM encompasses the period of inception to 2009. The brief chronology that follows highlights significant events and milestones of the park with emphasis on the planning and construction of the Gateway Arch. A primary reference source for this chronology was Sharon A. Brown's *Administrative History*.

- December 15, 1933: A Civic Committee is formed to investigate the establishment of a federal memorial honoring the Louisiana Purchase of 1803. Luther Ely Smith is named chairman.
- April 11, 1934: Jefferson National Expansion Memorial Association (JNEMA) is organized as a non-profit corporation.
- June 15, 1934: United States Territorial Expansion Memorial Commission is formed through a Joint Resolution of Congress.
- April 13, 1935: Executive Committee of the United States Territorial Expansion Memorial Commission approves the plan and scope for the federal memorial with an estimated cost of \$30,000,000.
- September 10, 1935: A City of St. Louis special bond issue is passed that authorizes up to \$7,500,000 in bond funds for construction of the federal memorial.
- December 21, 1935: Executive Order 7253 signed by the President to authorize the formation of JNEM and allocate \$6,700,000 in federal funds.
- May 23, 1938: Secretary of the Interior Harold Ickes insists on the removal of elevated tracks along the levee.
- October 10, 1939: Demolition of existing structures on the memorial site begun.
- May 1940: Department of the Interior accepts title to the Old Courthouse.
- November 1940: Julian Spotts succeeds John Nagle as superintendent of JNEM.
- May 1942: Demolition of all structures on the memorial site completed.
- January 29, 1945: Memorial design competition is announced by JNEMA.
- May 30, 1947: Memorial design competition is opened by JNEMA.
- February 18, 1948: Eero Saarinen and Associates announced as winner of the memorial design competition.
- May 25, 1948: The United States Territorial Expansion Memorial Commission approved the winning entry and recommended to the Secretary of the Interior that Saarinen's design be adopted.
- June 10, 1950: JNEM site dedicated by President Harry S. Truman.
- April 2, 1951: Luther Ely Smith dies.
- May 17, 1954: Public Law 361 (H.R. 6549) an Act authorizing the construction of the memorial in accordance with Saarinen's plan approved by the United States Territorial Commission and appropriating \$5,000,000.
- May 19, 1956: Supplemental appropriations bill signed to provide

\$2,640,000 for relocation of the elevated railroad.

- July 1, 1958: Amendment to the Act of May 17, 1954 for allocation of \$17,250,000 in funds for the construction of the entire JNEM project.
- February 1, 1959: George Hartzog, Jr., officially takes office as Superintendent of JNEM.
- June 23, 1959: Groundbreaking ceremony of the railroad relocation project.
- September 1, 1961: Eero Saarinen dies of brain tumor.
- March 14, 1962: MacDonald Construction Company awarded Gateway Arch and visitor center construction contract.
- June 27, 1962: Construction on the Arch begins with pouring of the concrete foundations.
- August 1, 1962: H. Raymond Gregg becomes Superintendent of JNEM.
- February 12, 1963: First stainless steel section of the Gateway Arch is set in place.
- May 1, 1963: St. Louis Hoist and Operation Engineers goes on strike and construction of the Arch is halted until May 27.
- July 1963: Creeper derricks installed on Arch legs.
- October 1963: Construction stopped on south leg due to failures in post-tension rods.
- June 23, 1964: Construction stopped on fabrication of stainless steel sections due to concerns raised by Richardson-Jordan, the structural engineer for Pittsburgh-Des Moines Steel, over structural stability in the upper half of the Arch.
- July 2, 1964: Construction of stainless steel sections restarted.
- July 14, 1964: Percy Green and Richard Daly, members of the Action Committee to Improve Opportunities for Negroes (ACTION), climb the Arch in protest of discriminatory hiring practices. Construction is stopped for the day.
- June 17, 1965: Stabilizing strut hoisted into place and installed between north and south legs.
- August 1, 1965: Leroy Brown assumes responsibility as Superintendent of JNEM, having earlier served as Assistant Superintendent with responsibility for overseeing construction.
- October 19, 1965: Public Law 89-269 authorizes the appropriation of an additional \$6,000,000 in funds.
- October 28, 1965: Installation of the final steel section, “topping out” ceremony, marking the completion of the Arch shell.
- January 7, 1966: The Building and Construction Trades Council unanimously voted to stop work on the visitor center over issues of hiring contractors not affiliated with the AFL-CIO. Controversy initiated when an African-American plumbing company was hired in an effort to meet federal equal opportunity requirements.

- February 9, 1966: Temporary injunction filed that requires union workers to return to work.
 - July 24, 1967: Inauguration of the north leg internal transportation tram.
 - March 19, 1968: Inauguration of the south leg internal transportation tram.
 - May 25, 1968: Dedication of the Gateway Arch by Vice President Hubert H. Humphrey.
 - August 10, 1976: Completion and public opening of the Museum of Westward Expansion; cost of construction \$3,178,000.
 - August 23, 1976: Dedication of the Museum of Westward Expansion.
 - 1998: The north and south tram load zones were extensively renovated to create additional space for museum displays. The three-story atrium was altered by the infill of the concrete flooring slab, removal of the concrete guard walls, and installation of carpeting over the terrazzo flooring.

Remodeling of the tram load zones initiated the installation of new interior branch circuit wiring, devices, and exit and emergency lighting fixtures. Incandescent lamps were removed and replaced with fluorescent lighting.
 - 2007: The electrical service was replaced due to an electrical accident that destroyed the original electrical service switchboard and necessitated the replacement of electrical service and upgrades to the fire and intrusion alarm systems.
 - 2009: Existing 235 KW and 300 KW diesel powered emergency generators removed and replaced by two 350 KW diesel powered emergency generators.
- Tram cable break and repair
 - Unknown: The original motor starters to air handling equipment were removed and replaced with variable frequency drives.

EVALUATION OF SIGNIFICANCE

Significance Criteria

The Criteria for Evaluation for listing on the National Register of Historic Places state:

The quality of significance in American history, architecture, archeology, engineering, and culture is present in districts, sites, buildings, structures, and objects that possess integrity of location, design, setting, materials, workmanship, feeling, and association, and:

- A. That are associated with events that have made a significant contribution to the broad patterns of our history; or
- B. That are associated with the lives of persons significant in our past; or
- C. That embody the distinctive characteristics of a type, period, or method of construction, or that represent the work of a master, or that possess high artistic values, or that represent a significant and distinguishable entity whose components may lack individual distinction; or
- D. That have yielded, or may be likely to yield, information important in prehistory or history.

Criteria Considerations

Ordinarily cemeteries, birthplaces, graves of historical figures, properties owned by religious institutions or used for religious purposes, structures that have been moved from their original locations, reconstructed historic buildings, properties primarily commemorative in nature, and properties that have achieved significance within the past 50 years shall not be considered eligible for the National Register. However, such properties *will qualify* if they are integral parts of districts that do meet the criteria or if they fall within the following categories:

- a. A religious property deriving primary significance from architectural or

artistic distinction or historical importance; or

- b. building or structure removed from its original location but which is primarily significant for architectural value, or which is the surviving structure most importantly associated with a historic person or event; or
- c. A birthplace or grave of a historical figure of outstanding importance if there is no appropriate site or building associated with his or her productive life; or
- d. A cemetery that derives its primary importance from graves of persons of transcendent importance, from age, from distinctive design features, or from association with historic events; or
- e. A reconstructed building when accurately executed in a suitable environment and presented in a dignified manner as part of a restoration master plan, and when no other building or structure with the same association has survived; or
- f. A property primarily commemorative in intent if design, age, tradition, or symbolic value has invested it with its own exceptional significance; or
- g. A property achieving significance within the past 50 years if it is of exceptional importance.⁸⁸

The Gateway Arch is significant under National Register Criterion C for its architectural and engineering design as well as for the role it played in the career of architect Eero Saarinen. The Gateway Arch is the focus of JNEM, for which the landscape was designed by Saarinen and noted landscape architect Dan Kiley. Saarinen and Kiley sculpted the surrounding landscape to create special views of the Arch.

⁸⁸ *Code of Federal Regulations, Title 36, Part 60*, "The National Register Criteria for Evaluation."

Eero Saarinen was born in Finland in 1910 and immigrated to the United States with his family in 1923. His father, renowned architect Eliel Saarinen, was the first president of the Cranbrook Institute of Architecture and Design in Bloomfield Hills, Michigan. After studying sculpture in Paris and Architecture at Yale University, Eero Saarinen joined his father's firm in 1937. Eero took over the firm in 1950 after his father's death.⁸⁹

In 1947, Eero Saarinen entered the architectural design competition for JNEM. His winning entry for what is now known as the Gateway Arch was one of the first major designs Saarinen completed on his own. Over the next thirteen years Saarinen designed several more influential projects, including the General Motors Technical Center outside of Detroit, the TWA Terminal at John F. Kennedy International Airport in New York City, and Dulles Airport outside of Washington, D.C. Saarinen died in 1961 at the age of 51, seven years before the Gateway Arch was formally dedicated.⁹⁰

Although some historians do not consider the Gateway Arch to be Saarinen's most influential design, others see it as his greatest contribution to American architecture. Architect Robert Venturi called the Arch "one of the best things since World War II—it is a thing that is very difficult to do which is to do a non-functional, sculptural symbolic gesture of enormous scale."⁹¹

⁸⁹ Laura Soullière Harrison, *National Register of Historic Places Nomination Form: Gateway Arch* (Washington, D.C.: NPS, 1985). This National Register nomination serves as documentation for the National Historic Landmark listing of the structure.

⁹⁰ Michael Capps, *Jefferson National Expansion Memorial, Architect Eero Saarinen*, viewed at <http://www.nps.gov/jeff/planyourvisit/architect.htm>, 2006.

⁹¹ As quoted in Soullière Harrison, 7.

The design of the Gateway Arch is based on a weighted catenary; however, neither the extrados nor the intrados of the arch is a true catenary. The arch is constructed of prefabricated double-wall carbon steel and stainless steel triangular segments that reduce in size as they approach the apex. This stressed metal double skin carries the structural loads, eliminating the need for interior framing. The innovative structural engineering design of the Arch by Hannskarl Bandel and Fred Severud contributes to its significance. In addition, the inventive tram system within the legs of the Arch that lifts visitors to the top of the Arch is a significant feature.

While properties that are primarily commemorative in intent are often not considered eligible for the National Register, Criteria Consideration F states that such a structure can be listed if design, age, tradition, or symbolic value has invested it with its own exceptional significance.⁹² Such is the case with the Gateway Arch, as its innovative design and symbolic value have made it an important icon of the built American landscape. Since the beginning of its construction in the 1960s, the Gateway Arch has been a symbol of the city of St. Louis and its role as the "gateway to the West."

Properties that have achieved significance within the past fifty years are also generally not considered eligible for inclusion on the National Register. However, Criteria Consideration G asserts that such properties can be listed if they are of exceptional importance. The groundbreaking design of the Gateway Arch allowed it to achieve significance almost immediately, as evidenced by its addition to the National Register in 1977

⁹² CFR 36, "The National Register Criteria for Evaluation."

and its designation as a National Historic Landmark in 1987.⁹³

Consideration of significance to the level of a National Historic Landmark is discussed in the National Register Bulletin, *How to Prepare National Historic Landmark Nominations*

By definition, the almost 2,300 properties designated as National Historic Landmarks are the most significant places in American history—they illustrate and commemorate our collective past and help us to understand our national identity. National Historic Landmarks outstandingly represent and interpret the best and brightest and the most tragic aspects of our history. Through these landmarks, all Americans can better understand and appreciate the broad trends and events, important persons, great ideas and ideals, and valuable accomplishments in the arts and sciences, and humanities, that are truly significant in our history.⁹⁴

Potential National Historic Landmarks are evaluated for their national significance according to a set of criteria that is different from the more familiar National Register criteria. The Criteria for Evaluation for the designation of a National Historic Landmark (NHL) state:

The quality of national significance is ascribed to districts, sites, buildings, structures, and objects that possess exceptional value or quality in illustrating or interpreting the heritage of the United States in history, architecture, archeology, engineering, and culture and that possess a high degree of integrity of location, design, setting, materials, workmanship, feeling, and association, and:

- A. That are associated with events that have made a significant contribution to, and are identified with, or that outstandingly

represent, the broad national patterns of United States history and from which an understanding and appreciation of those patterns may be gained; or

- B. That are associated importantly with the lives of persons nationally significant in the history of the United States; or
- C. That represent some great idea or ideal of the American people; or
- D. That embody the distinguishing characteristics of an architectural type specimen exceptionally valuable for a study of a period, style or method of construction, or that represent a significant, distinctive and exceptional entity whose components may lack individual distinction; or
- E. That are composed of integral parts of the environment not sufficiently significant by reason of historical association or artistic merit to warrant individual recognition but collectively compose an entity of exceptional historical or artistic significance, or outstandingly commemorate or illustrate a way of life or culture; or
- F. That have yielded or may be likely to yield information of major scientific importance by revealing new cultures, or by shedding light upon periods of occupation over large areas of the United States. Such sites are those which have yielded, or which may reasonably be expected to yield, data affecting theories, concepts and ideas to a major degree.

National Historic Landmark Exclusions

Ordinarily, cemeteries, birthplaces, graves of historical figures, properties owned by religious institutions or used for religious purposes, structures that have been moved from their original locations, reconstructed historic buildings and properties that have achieved significance within the past fifty years are not eligible for designation. If such properties fall within the following categories they may, nevertheless, be found to qualify:

⁹³ Soullière Harrison, 8.

⁹⁴ National Register Bulletin, *How to Prepare National Historic Landmark Nominations* (Washington, D.C.: National Park Service, 1999).

1. A religious property deriving its primary national significance from architectural or artistic distinction or historical importance; or
2. A building or structure removed from its original location but which is nationally significant primarily for its architectural merit, or for association with persons or events of transcendent importance in the nation's history and the association consequential; or
3. A site of a building or structure no longer standing but the person or event associated with it is of transcendent importance in the nation's history and the association consequential; or
4. A birthplace, grave or burial if it is of a historical figure of transcendent national significance and no other appropriate site, building, or structure directly associated with the productive life of that person exists; or
5. A cemetery that derives its primary national significance from graves of persons of transcendent importance, or from an exceptionally distinctive design or an exceptionally significant event; or
6. A reconstructed building or ensemble of buildings of extraordinary national significance when accurately executed in a suitable environment and presented in a dignified manner as part of a restoration master plan, and when no other buildings or structures with the same association have survived; or
7. A property primarily commemorative in intent if design, age, tradition, or symbolic value has invested it with its own national historical significance; or
8. A property achieving national significance within the past 50 years if it is of extraordinary national importance.⁹⁵

⁹⁵ *National Register Bulletin 16a: How to Complete the National Register Registration Form* (Washington, D.C.: NPS, National Register of Historic Places, 1977, revised 1997).

The Gateway Arch was designated as a National Historic Landmark on May 28, 1987, and was documented in the National Historic Landmark theme study, "Architecture in the Parks." The properties included in this thematic nomination are nationally significant for their architecture, are located within the boundaries of an area of the National Park system, and were constructed for visitor use or for interpretive or administrative purposes. JNEM was the first major national park development after World War II. The design of the Gateway Arch was a turning point in the shift from the rustic style of park architecture used for buildings throughout the 1920s and 1930s to a more modern style of architecture that characterized the Mission 66 period.⁹⁶

Period of Significance

As a structure considered primarily significant for its architectural design, the period of significance for the Arch is associated with its initial design and construction. The relatively minor changes to the Arch since its completion in 1965 are not considered to be contributing alterations. Therefore, the period of significance is dated to the official dedication of the Arch in May 1968. The visitor center, including the lobby and museum, were excluded from the scope of study of this report.

⁹⁶ Soullière Harrison, *National Park Service: Architecture in the Parks*, <www.nps.gov/history/history/online_books/harrison/harrison0.htm>, 1986. Refer also to Ethan Carr, *Mission 66: Modernism and the National Park* (Amherst: University of Massachusetts Press, 2007). According to Carr, the proposed design of the Gateway Arch was the turning point in NPS architecture. It influenced the decision to use modern architecture for the Mission 66 program, as it proved that a modern design could be successful at "interpreting" the park to contemporary visitors, compared to earlier rustic-style park buildings which sought to blend with the natural and vernacular character of the park.

Character-Defining Features

The historic nature of significant buildings and structures is defined as their character, which is embodied in their identifying physical features. Character-defining features can include the shape of a building; its materials, craftsmanship, interior spaces, and features; and the different components of its surroundings.⁹⁷

The most important character-defining feature of the Arch, whose design is based upon a weighted catenary, is the simple arch shape itself. The modulation of the shape, which tapers in cross section from grade to the apex of the Arch, as well as the overall height and breadth of the shape, were carefully studied and repeatedly refined by Saarinen as the design evolved. The overall shape of the Arch gives the structure its visual identity. The arch shape is also a key component of the symbolic intent of the memorial design, which is to commemorate the “Gateway to the West.” The overall shape is defined by the metal skin of the Arch, which is the load-bearing element of the structure. The stressed metal skin allows the interior of the Arch to be free of large-scale framing or reinforcement. Since the exterior skin is also the primary structure of the Arch, the overall shape of the exposed exterior surface gains added importance as a character-defining feature.

While difficult to see from afar, the thirty-two small openings at the top of the structure are an important aspect of the Arch. Saarinen envisioned a space at the apex of the Arch from which visitors could view the

surrounding area; thus these openings are important to the function of the arch.

The stainless steel material that covers the exterior of the Arch contributes to the overall character of the structure. The reflectivity of the material is an important aspect of the Arch’s design. At close range, the craftsmanship of the machined finishes of the stainless steel further adds to the overall character of the Arch.

Although much of the interior volume of the Arch itself has a utilitarian character, there are some individual spaces that are important to visitors’ experience of the Gateway Arch. The observation level at the apex of the Arch was an important part of Saarinen’s initial concept for the memorial. Other important spaces include the north and south tram load zones at the base of the Arch. The spatial volume of these underground areas as designed by Saarinen and Associates is a notable character-defining feature of the Arch interior. Although not generally accessible to the public, the egress stairs positioned within each leg of the Arch are a structurally distinctive element that can be viewed through the windows of the tram.

The tram by which visitors ascend and descend the Arch is also a character-defining element of the structure, as well as a unique engineering invention. The custom-built tram system was specifically designed to allow visitation to the top of the uniquely shaped Arch structure, and therefore it is a critical part of the intended function of the memorial. The tram ride is a key component of the visitor’s experience in the Arch.

The interiors of the Arch and the museum and visitor center are characterized by a simple and limited palette of materials. The care given to the interior design and procession of a visitor’s experience is significant. Saarinen

⁹⁷ Lee H. Nelson, FAIA, *Preservation Brief 17: Architectural Character: Identifying the Visual Aspects of Historic Buildings as an Aid to Preserving Their Character* (Washington, D.C.: NPS, Technical Preservation Services, 1988).

allows the visitor to pass by the base of the Arch before descending inside the visitor center. A progression of ramps, exhibit spaces, and stairs leads to the foundations of the Arch, where visitors board the tram to reach the observation level.

Although the conclusions of this study indicate that interior spaces and materials can be considered historically significant, the current National Register documentation does not discuss or specifically designate interior spaces as significant, nor has any interior space been officially determined to be eligible for National Register listing. In recent years, interior architectural features have been changed to meet contemporary park needs or to achieve code compliance. The park is currently in discussion with the Missouri State Historic Preservation Office (SHPO) regarding evaluation of the historic significance of the interior finishes and spaces.

Several aspects of the setting of the Gateway Arch are important to the visual character. The surrounding landscape designed by Saarinen and Dan Kiley adds significantly to the Arch's overall character. The Arch's proximity to the river as well as its placement on axis with the Old Courthouse are also important to the visual character of the Arch and underscore its historical/commemorative function.

Another important aspect of the Gateway Arch is the sequence of approach to the memorial. Eero Saarinen and Dan Kiley envisioned the Arch as emerging from a forest-like green space juxtaposed with the surrounding urban landscape. The surrounding landscape was designed with views of the Arch in mind. These views as well as the sequence of approach experienced by the visitor add significantly to the character of the Gateway Arch itself.

Assessment of Integrity

Assessment of integrity is based on an evaluation of the existence and condition of the physical features which date to a property's period of significance, taking into consideration the degree to which the individual qualities of integrity are present. The seven aspects of integrity as defined in the National Register Criteria for Evaluation are location, design, setting, materials, workmanship, feeling, and association. As noted in *National Register Bulletin 15: How to Apply the National Register Criteria for Evaluation*:

Location is the place where the historic property was constructed or the place where the historic event occurred. . . . Design is the combination of elements that create the form, plan, space, structure, and style of a property. . . . Setting is the physical environment of a historic property. . . . Materials are the physical elements that were combined or deposited during a particular period of time and in a particular pattern or configuration to form a historic property. . . . Workmanship is the physical evidence of the crafts of a particular culture or people during any given period in history or prehistory. . . . Feeling is a property's expression of the aesthetic or historic sense of a particular period of time. . . . Association is the direct link between an important historic event or person and a historic property.⁹⁸

For NHL designation, a property should possess these aspects to a high degree. The property must retain the essential physical features that enable it to convey its historical significance. The essential physical features are those features that define both why a property is significant (NHL criteria and themes) and when it was significant (period of significance). National Register Bulletin 15

⁹⁸ *National Register Bulletin 15: How to Apply the National Register Criteria for Evaluation*, 44–45.

defines integrity as “the ability of a property to convey its significance.”⁹⁹ As noted in the National Register Bulletin, *How to Prepare National Historic Landmark Nominations*,

Integrity is the ability of a property to convey its historical associations or attributes. The evaluation of integrity is somewhat of a subjective judgment, but it must always be grounded in an understanding of a property’s physical features and how they relate to its historical associations or attributes. The NHL survey recognizes the same seven aspects or qualities of integrity as the National Register.¹⁰⁰

The primary historical significance of the Gateway Arch is related to the innovative design of the structure. The integrity of the Arch itself as well as the integrity of other original features of Saarinen’s concept, such as the connection between the site and the Arch, are the most important physical aspects that convey this significance. The discussion below considers each of the seven aspects of integrity as they relate to the Gateway Arch.

Integrity of Location. The Gateway Arch retains a high degree of integrity of location in relationship to its site. The building location and the boundaries of the site are unchanged since the Arch was dedicated in 1968.

Integrity of Design. The Gateway Arch retains a high degree of integrity of design, as few alterations to the structure have been implemented since its original construction. While minor alterations have been made to the interior of the Arch, including flooring infill of portions of the two-story tram load zones, the initial design concept by Kevin Roche of Saarinen and Associates is still evident.

Integrity of Setting. The Gateway Arch retains a high degree of integrity of setting. The adjacent parkland also retains a high integrity of setting as the surrounding environment reflects the environment as it existed when the park was completed. By 1968, the broader urban context around the memorial already included the characteristic elements of the city today, including high rise commercial buildings, the Mississippi River bridges, and the nearby railroads and interstate highways. Saarinen’s initial concept of having the arch emerge from an open green space within an urban setting is still present today.¹⁰¹

Integrity of Materials and Workmanship. The Gateway Arch retains a high degree of integrity of materials and workmanship. The exterior retains its original materials, and the majority of the exterior surface has been left untouched since the initial construction of the Arch. Some of the interior materials have been changed. This has included replacement of worn flooring materials in public areas, to match the original materials. Although the original materials have been replaced, the replica materials installed are similar mass-manufactured materials. In other areas such as the tram load zones, new wall cladding and interpretive displays have been added over the original wall surfaces, which remain intact although concealed. These interior changes have a minor impact on the integrity of the Arch.

Integrity of Feeling. The Gateway Arch retains a high degree of integrity of feeling. The structure still conveys the historic and aesthetic feeling of a symbolic gateway and a public memorial as was the original design intent.

⁹⁹ Ibid.

¹⁰⁰ National Register Bulletin, *How to Prepare National Historic Landmark Nominations* (Washington, D.C.: National Park Service, 1999).

¹⁰¹ Regina M. Bellavia and Gregg Bleam, *Cultural Landscape Report* (Omaha, Nebraska: NPS, 1996), 188–190.

Integrity of Association. The Gateway Arch retains a high integrity of association. The Arch and JNEM as a whole continue the commemoration of the westward expansion of the United States in the nineteenth century. The Arch also retains its association with Eero Saarinen as his original design intent is still evident today.

PHYSICAL DESCRIPTION AND CONDITION ASSESSMENT

An inverted catenary curve, rising 630 feet above grade with massive concrete foundations extending nearly 50 feet below grade, and with an overall width equal to its height, the Gateway Arch exemplifies the intricacy of structure within the confines of one of the simplest building forms, the arch. An orthotropic design, in which the inner and outer skins are attached together to form a composite structure, was utilized for the Arch; thus an internal structural skeleton does not exist.

Each leg of the Arch is constructed of double-walled equilateral triangular “tube” segments, with an outer dimension of each side of the triangular section measuring 54 feet at grade level and tapering to 17 feet at the top. These dimensions are taken from original structural drawings, which are referenced throughout this section of the report. To an elevation of 300 feet above grade, the interstitial space between the inner and outer walls or skins of the double-walled segments is filled solid with reinforced concrete, utilizing post-tensioning bars in the extrados corners to provide stiffness and dead weight to the arch to resist overturning, bending and torsional moments caused by wind, temperature change, and construction loading of the creeper derricks and to provide rigidity during construction.

The space between the inner and outer walls at the base is 3 feet 4-1/2 inches at the termination of the concrete fill and only 7-3/4 inches above the 400 foot level; this dimension remains consistent to the top of the Arch. The outer skin assembly consists of 1/4 inch thick stainless steel plates welded together, while the interior skin assembly consists of 3/8 inch thick type A-7 carbon steel plates, except at the corners where the plates are 1-3/4 inches thick (Refer to detail

“Section 3 at Corners” on Sheet S-107 in Appendix I).

A field investigation was conducted as part of the condition assessment for this study. The assessment is based only on visual observations of representative sections of accessible areas of the structure and review of available drawings, specifications, reports, photographs, previous structural evaluations, and other archival materials. Information from the visual assessment and document review was supplemented by an oral interview with Mr. Kenneth Kolkmeier, Project Manager for Pittsburgh Des Moines Steel Company (PDM), on January 14, 2009. Visual observation included review of both interior and exterior structural elements. Many of the observations made from the exterior were performed via binoculars, because of limitations of vertical access on the exterior. The field observation and structural evaluation were completed by representatives of BVH, WJE, and Alvine during November 2008.

Exterior

The exterior assessment of the Arch as presented in this report draws upon limited field review performed as part of the HSR study as well as the focused investigation conducted by the project team for the *Gateway Arch Corrosion Investigation, Part I*, dated May 2006. The exterior of the Arch was observed from the dome of the Old Courthouse and from grade. Binoculars and spotting scopes were used to view the structure and assist with mapping of discoloration and other distress. Each side of the north and south legs, as well as the top of the structure, were inspected; given the available vantage points, certain areas, such as the upward-facing surfaces of the low-numbered Arch segments, could not be viewed. A close up visual inspection of the lowest ten feet, including sounding for delamination, was performed on the exterior skin of the north and south legs of the Arch. This inspection was made from the plaza at the base of the legs.

Areas of staining were categorized as

- 1) Severe, staining could be seen easily from the ground without any instruments;
- 2) Moderate, staining could be seen from the ground with careful observation without any instruments;
- 3) Light, staining could be seen with binoculars;
- or 4) Very Light, staining could be seen with binoculars with careful inspection.

Additionally, vertical staining, which involves lighter or darker vertical lines through the staining, and dark area or patches of staining were observed. See Appendix G for a graphic summary of the results of the condition survey conducted as part of the *Gateway Arch Corrosion Investigation, Part I*.

This section contains a description of exterior conditions observed during the study; refer to the Structural section below for further discussion of causes of observed distress.

Staining and Discoloration

The south face of the south leg and the north face of the north leg were viewed from the ground. These two locations appear to be in good condition, with little corrosion or staining. Most of the staining appears to follow the path of rain run-off of the welds that were applied in the field. The construction creeper crane rails were mechanically attached to these locations during construction of the Arch. The lower locations of the removed anchors are visible from the ground. The literature review suggests these surfaces were carefully cleaned as the derrick rails were being removed. See Figures 33 through 35 showing the south face of the south leg, with conditions that are also typical of the north face of the north leg.

On the east and west faces of the Gateway Arch, the staining is more severe. The majority of the staining is rust-colored and appears at levels above the 300 foot concrete fill. From these observations, it is hypothesized that corrosion has initiated at a contaminated or damaged surface. The west faces of both the north leg and the south leg were viewed from the dome of the Old Courthouse and from the ground (Figure 36). The east face of both legs was viewed from the ground (Figures 34 through 39).

The stainless steel skin on the topside of the structure was inspected from the roof hatch. The stainless steel appears to be in good condition. Figures 40 and 41 show the stainless steel panels and welds from the roof hatch.



Figure 33. Staining, deposits, and corrosion on south face of south leg due to rain run-off at field welds. Source: WJE, 2008.

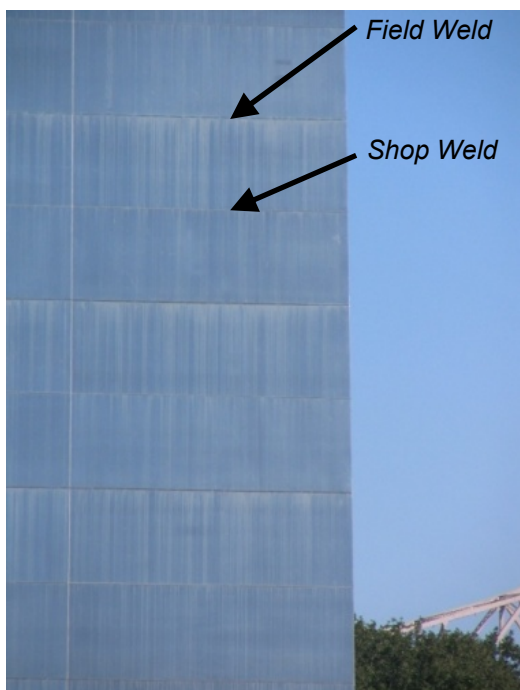


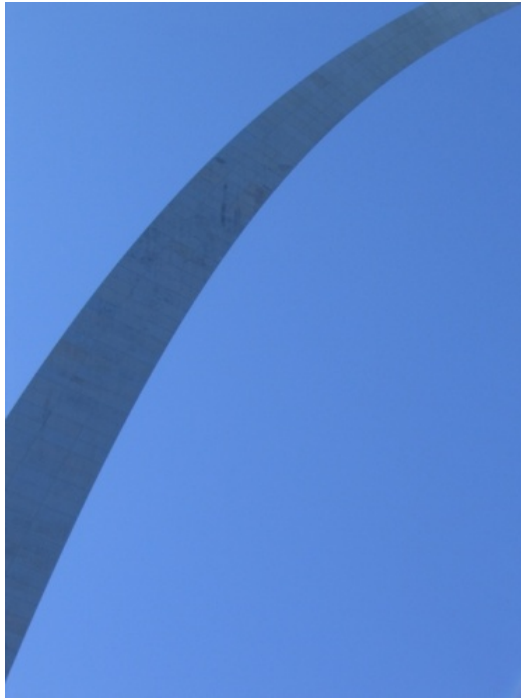
Figure 34. Closer view showing run-off at field welds. Source: WJE, 2008.



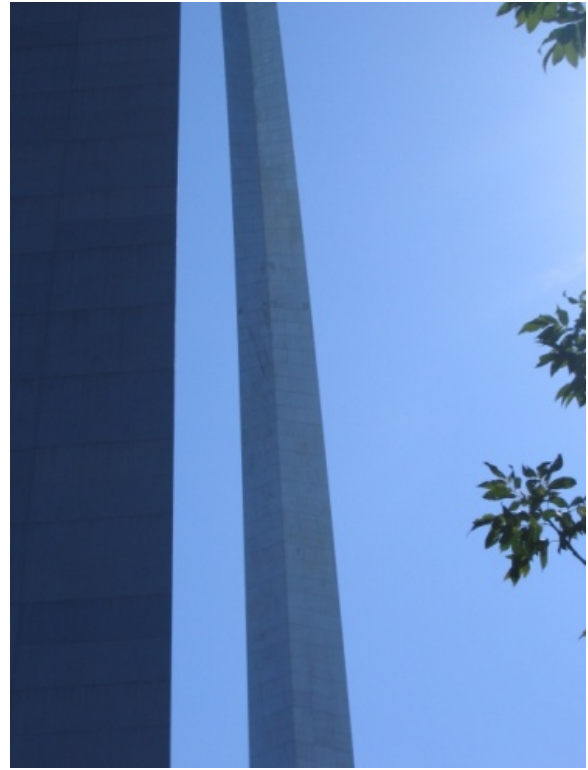
Figure 35. Overview of the south face of the south leg. A change in coloration is discernible at the 300 foot level near the top of the image. Source: WJE, 2008.



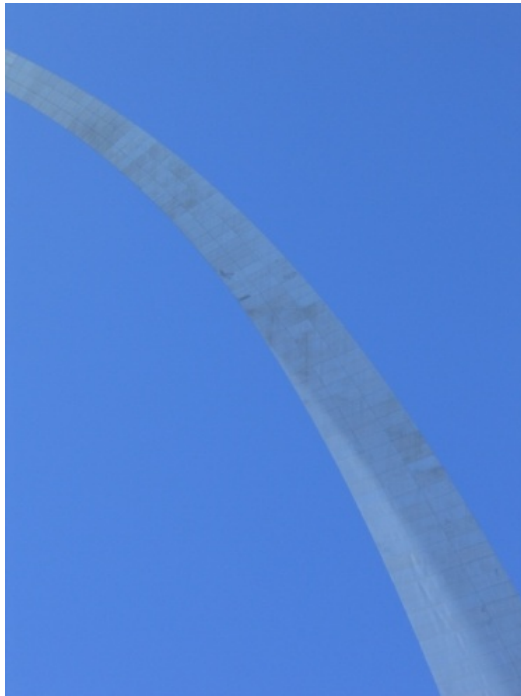
Figure 36. Overview of the west face of the north leg. Source: WJE, 2008.



*Figure 37. Overview of the east face of the south leg.
Source: WJE, 2008.*



*Figure 39. View of the south leg seen from grade,
with the north leg in the foreground. Source: WJE,
2008.*



*Figure 38. Overview of the east face of the north leg.
Source: WJE, 2008.*



*Figure 40. View looking south from the roof hatch at
the top of the Arch. Source: WJE, 2008.*



Figure 41. Closer view of stainless steel panels and welds at the roof hatch at the top of the Arch. Source: WJE, 2008.

During the brief inspection of the exterior stainless steel skin of the structure from the roof hatch at the top of the Gateway Arch, samples of the corrosion or staining were obtained using adhesive tape. The samples taken did not yield particles of corrosion, staining, or steel material large enough for analysis. Removal of larger samples or metal coupons was outside the scope of this study.

Streaking, caused by the field welds, is obvious on the north and south faces. The field welds are not ground flush, thus have more roughness that collects atmospheric soiling that washes out when water runs over the welds, causing streaking below (Figures 42 and 43). Streaking caused by the field welds is also detectable on the east and west faces (Figure 44). Due to the curved shape of the Arch, the streaking on the east and west faces often flows diagonally to the grid pattern of the stainless steel panels.

Locations of bearing pads beneath creeper derrick rails are visible. The pads may have polished or abraded the surface of the stainless steel panels, leaving lasting marks (Figure 45).



Figure 42. Overview of the south face of the south leg showing streaking originating at field welds. Source: WJE, 2008.

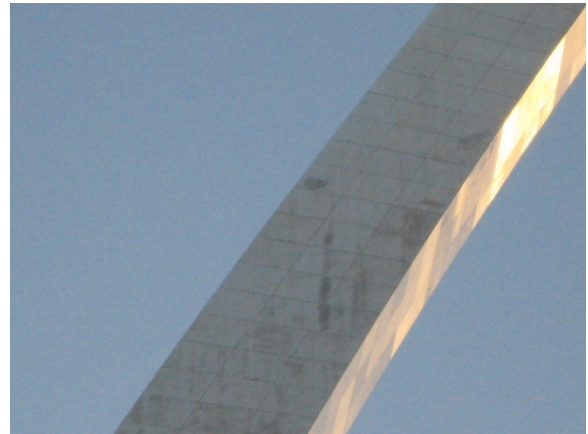


Figure 43. Dark staining on the east face of the south leg, showing dark staining originating at field welds. Source: WJE, 2008.

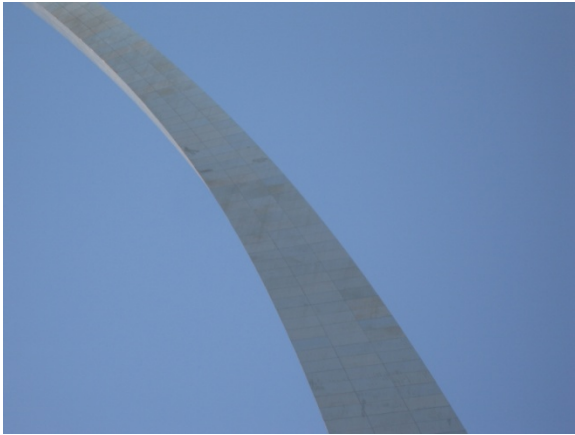


Figure 44. Overview of intermittent staining on the east face of the north leg. Source: WJE, 2008.



Figure 45. Detail showing mark on Arch surface at the location of an original derrick creeper crane bearing pad. Source: WJE, 2008.

Graffiti and Vandalism

Approximately the lower fifteen feet of exterior surface at the base of each leg have superficial brown staining and mechanical surface damage. The surface has been damaged by graffiti and gouging in the steel with sharp tools, hammers, drills, etc. Also, previous abrasive cleaning techniques used to remove graffiti may have contributed to the existing superficial staining. Some of the corrosion at the base can be attributed to the presence of de-icing salts. Park maintenance staff described the use of salts during winter months around the base of the Arch and along entry routes to the Arch visitor center. Given the recent installation of a snow-melt system and new paving around the base of each leg, it would be expected that further use of de-icing salts will be limited. Additionally, some panels above the base level are also corroded where de-icing salts would usually not be present. See Figures 46 through 49 for representative examples of the steel condition at the base of each leg.

The lowest two sections near the grade of the arch exhibit incised graffiti and corrosion staining. The incised graffiti ranges in condition from shallow scratching (Figure 50) to deep scratching. At some locations there is red corrosion associated with the scratches (Figure 51), as further discussed below. In addition, horizontal corrosion stain lines were observed near the base that may be related to the possible use of steel shovels or plows used to remove snow from the plaza in the winter (Figure 52). The plaza has a recently installed snow melt system, and the future use of metal shovels will be limited. At some locations near the base there are also light red corrosion stains (Figure 53).

When stainless steel oxidizes, a thin passive complex chromium oxide layer forms on the surface. Stainless steel may have a dark brown

corrosion product when “the environment overwhelms the stainless steel’s passive film and it cannot heal the interruption.”¹⁰² As a result, in crevices and pits a red oxidation product may form as was observed on the Gateway Arch at the incised graffiti. Ferrous deposits from the snow-removal equipment or the implements used to create the incised graffiti may also be contributing to the red corrosion stains. The presence of aerosol chlorides from deicing salts may also have contributed the pitting corrosion of the stainless steel. The newly-installed snow-melt system is expected to limit future use of deicing salts. Atmospheric contaminants and pollutants may also contribute to surface corrosion of the stainless steel.



Figure 46. Graffiti damage at the base of the Arch. Source: WJE, 2008.

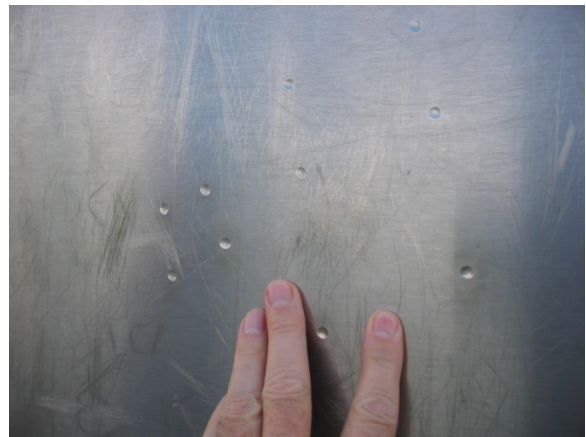


Figure 47. Mechanical damage, likely due to vandalism, at the base of the Arch. Source: WJE, 2008.



Figure 48. Detail of brown staining near the base of the Arch, which forms at the locations of scratches or other mechanical damage. Source: WJE, 2008.

¹⁰² *Designer Handbook: Stainless Steel for Coastal and Salt Corrosion*. Specialty Steel Industry of North America, <www.ssina.com/architect/publications/salt_corrosion.html>, accessed May 27, 2010.



Figure 49. Periodic cleaning to remove graffiti is changing the finished surface of the panels near the base of the leg. Source: WJE, 2008.

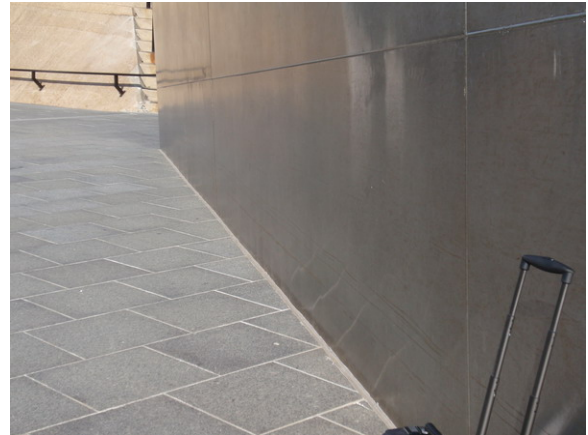


Figure 52. Red-colored staining and scratches near grade, possibly associated with past use of steel snow-removal equipment. Source: WJE, 2008.



Figure 50. Scratches and incised graffiti on the Arch. Source: WJE, 2008.



Figure 53. Light red-colored staining at scratches near grade. Source: WJE, 2008.

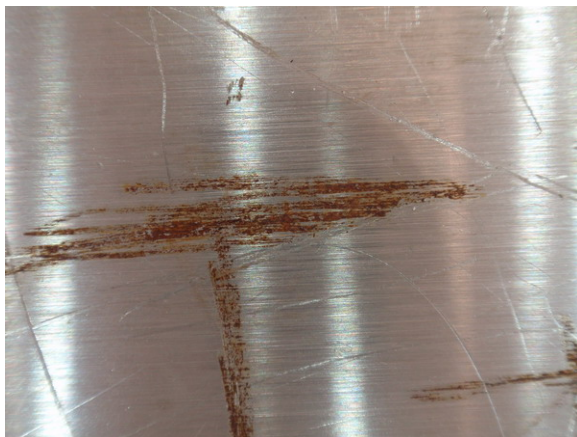


Figure 51 Red-colored corrosion associated with scratches. Source: WJE, 2008.

Visual Distortions

Visual distortion of the surface of the Arch is more visible above the concrete infill, which is consistent with conditions documented in archival photographs (Figures 54 through 56). One apparent phenomenon on the Arch is “oil canning,” which refers to the tendency of flat sheets of metal to undulate in and out of plane. The lack of optical flatness is more apparent on reflective surfaces and appears as a slight darkening and waviness. Oil canning is visually prominent above the 300 foot level where the stainless steel is reinforced on the inside with steel stiffeners. Below the 300 foot level, where concrete was introduced into the interstitial space, minimal oil canning is apparent. This phenomenon was apparent soon after the Arch was completed, according to Mr. Kolkmeier.¹⁰³

The variations in surface plane (waviness) of the exterior plates is less visible on the north and south faces of the legs, as compared with the east and west, because of the specific geometries at these locations. The plate buckling characteristics are visible on the north and south faces in certain lights (Figures 57 and 58). Regarding the wrinkles in the exterior plates, Mr. Kolkmeier noted that Section 46 on the north leg buckled slightly as it was being worked into place, and that PDM dismantled and reworked the steel plates in this area. He also mentioned that “anyone who expected that it [the 1/4 inch stainless steel panels] would have no wrinkles did not have a very good instruction in strength of material because when you weld on the back side of 1/4 inch material you get distortion from the welding.”¹⁰⁴ Mr. Kolkmeier also noted that structural engineer Hannskarl Bandel helped develop the solution implemented at Section 46; holes were cut in the top of



Figure 54. View of oil canning of steel panels on the east face of the north leg. Source: WJE, 2008.



Figure 55. View of oil canning on the west face of the north leg. Source: WJE, 2008.

¹⁰³ Kolkmeier, interview by Worth et al., January 14, 2009.

¹⁰⁴ Ibid.



Figure 56. View of oil canning on the east and west faces of the south leg near grade. Source: WJE, 2008.

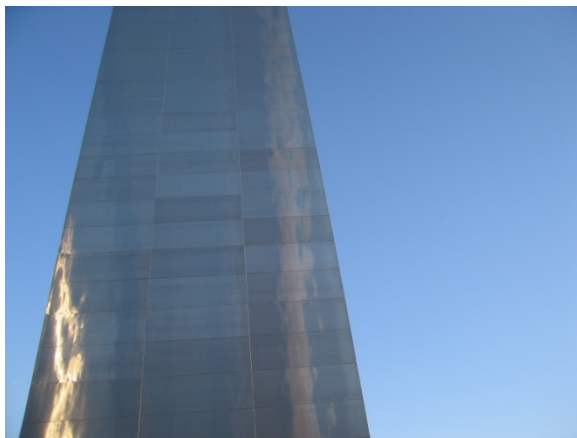


Figure 57. Surface waviness on the outside face of the Arch leg. Source: WJE, 2008.



Figure 58. Surface waviness on the west face of the south leg, visible in late afternoon sunlight. Source: WJE, 2008.

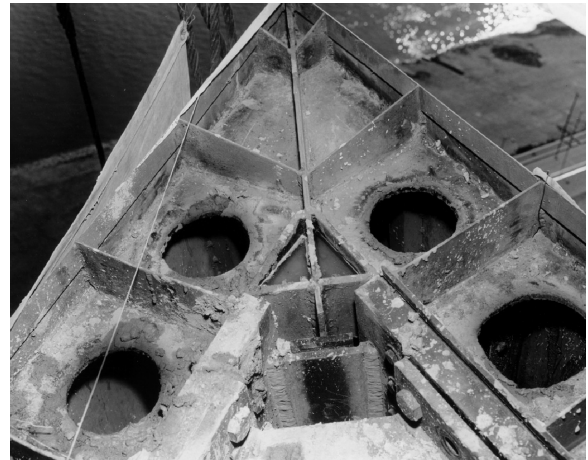


Figure 59. View of the continuous transverse plates added to Segment 46. Photograph courtesy of Ken Kolkmeier.

the continuous transverse plates and the section was filled with concrete, as shown in Figure 59. At the south leg Section 46 was also filled with concrete to provide symmetry. The outlines of the vertical stiffener angles are visible above and below the level of the concrete fill (Figure 60).

Stitch welds of vertical stiffener angles below the level of the concrete fill are slightly visible on the exterior skin in certain lights (Figures 61 through 63). Stitch welds of vertical stiffener angles above the level of the concrete fill are more significantly visible on surface of the exterior skin.

The so-called “grease spots” are spots that are apparently related to the connection of the horizontal strut truss. It is possible that the filling and hand-finishing of the truss contact points is rougher and therefore more likely to attract and hold dirt and other debris. Based on archival photographs, these stains were not obvious when the Arch was completed. The hand repairs to the surface may have left the surface prone to dirt collection. In addition oil contamination left by construction operations may have attracted dirt over time. Close-up observations are needed to confirm this condition.

Adjustments were made to the design and materials of the Arch after the bidding process, for budget purposes. These adjustments included reducing the exterior stainless steel plate thickness from 7/16 inch to 1/4 inch and reducing the thickness of the mild steel plate from 1/2 inch to 3/8 inch. This could account for some of the surface variations observed at the weld locations. Mr. Kolkmeier indicated that these changes made the engineers “a bit nervous.”¹⁰⁵

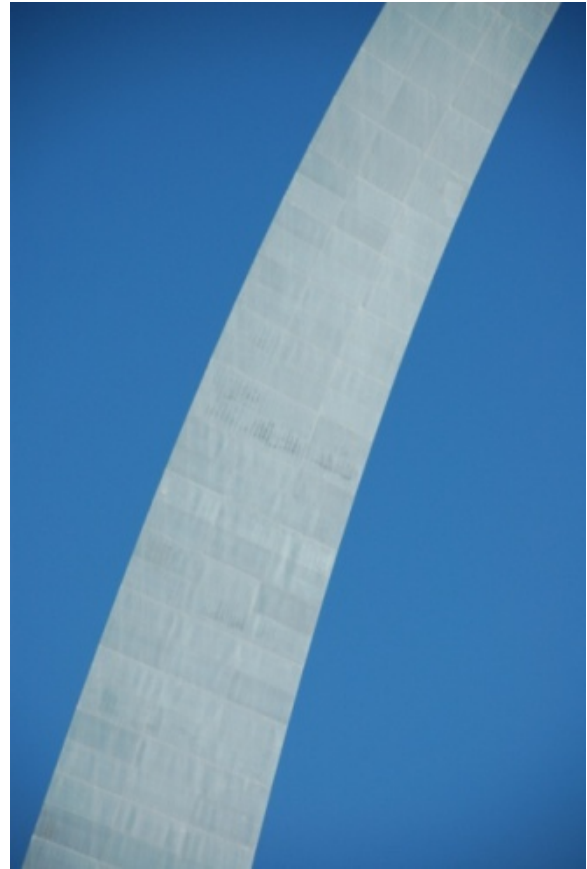


Figure 60. View of the north leg at the 300 foot level. Source: WJE, 2008.

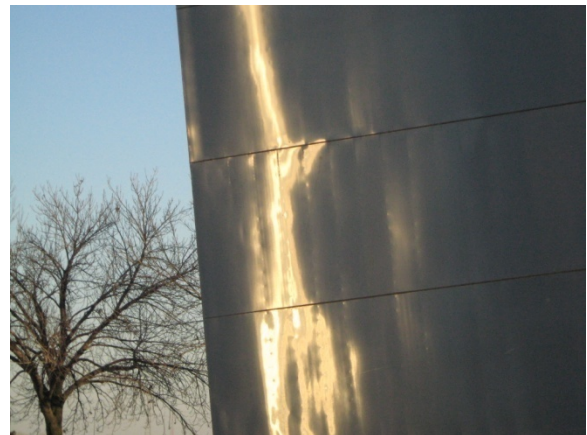


Figure 61. Vertical lines corresponding to stiffener angles are visible near grade. Source: WJE, 2008.

¹⁰⁵ Ibid.



Figure 62. Vertical lines corresponding to stiffener angles are visible below the 300 foot level. Source: WJE, 2008.



Figure 63. Vertical lines corresponding to stiffener angles are visible near grade on the west face of the south leg. Source: WJE, 2008.

Interior

The interior of the Gateway Arch is characterized by multiple interconnected spaces that flow continuously from the entrance of the visitor center to the tram queuing area that is located in the lowest portion of the Arch. Visitors enter the Arch complex at the base of the Arch legs located at ground level and proceed to the subterranean visitor center via ramps. The ramps turn back and continue to the tram load zones north and south of the visitor center. The north and south legs are identical and symmetrical with regard to spatial arrangement. Each tram load zone is composed of a large three-story space located at the base of the Arch legs with upper, lower, and queuing levels that are connected spatially. The upper level is dedicated to interpretive displays and overlooks the lower level and queuing level. The upper and lower levels are connected by stairs. The lower level overlooks the queuing level. Visitors proceed from the lower level to the queuing lines, which descend to the lowest level of the tram load zone via a sequence of terrazzo stairs and a landing at each tram capsule.

Each leg of the Arch has eight tram capsules that carry visitors to the observation level. The eight tram capsule openings are spaced evenly across the queuing level with each tram capsule entrance lower than the next. As the tram ascends the leg of the Arch, each capsule continuously levels to the curvature of the Arch's catenary curve. The trams deposit visitors at the tram load zone located at the top of the Arch. Visitors disembark from the trams and proceed up a sequence of metal plate stairs and landings to the observation level. The observation level floor curves with the catenary curvature of the Arch and connects the legs of the Arch, forming a continuous path.

North and South Tram Load Zones

Walls

The walls of the north and south tram load zones consist of exposed grey concrete with vertical form board markings. Vertical board formwork was used extensively, especially in the original construction of the subterranean portions of the Gateway Arch. The graining and knot defects of the formwork are translated to the exposed concrete walls, providing subtle texture to the walls. The walls of the tram load zones are generally of a smooth finish with sporadic indication of voids and honeycombing where air pockets formed during the pours. The walls exhibit minor spalling and chipping of the surface paste, revealing the aggregate beneath (Figure 64). The lift pours of the concrete walls are evident and occur approximately every 10 feet vertically. The seams of the pour lifts have been troweled over with a cementitious patching material, and vertical grooves cut to continue the vertical formboard markings of the walls. The horizontal line of the lift demarcations is flush with the rest of the wall surface; however, the patch material is of a different coloration and texture, creating a banding effect (Figure 65). Horizontal cracks have developed at the seams of the pour lifts, most likely indicating a cold joint that has telegraphed through the cementitious patching. Minor cracks have developed in the concrete walls, possibly due to shrinkage of the concrete. These cracks range in size from hairline to approximately 1/16 inch in width (Figures 66 and 67). At isolated locations, foreign objects can be seen embedded in the concrete wall; a tongue-and-groove-wood slat is embedded in the concrete wall as shown in Figure 68.



Figure 64. Example of concrete wall surface spalling in the lower tram load zones. Source: BVH, 2008.



Figure 65. Demarcation of concrete pour lifts in lower tram load zones. Note the hairline cracking at cold joint between lifts. Source: BVH, 2008.



Figure 66. Hairline crack in concrete wall. Note previous attempt at patching. Source: BVH, 2008.

While generally in fair condition, the walls show signs of moderate to severe deterioration in a few locations. Indications of water infiltration in certain areas have caused severe staining to the concrete walls. These areas are near expansion joints and wall-to-ceiling interfaces. The most extensive water infiltration occurs at the west wall of the stairs in the north leg of the Arch that lead down to the lower tram load zone. The area of leakage occurs at the location of an expansion joint in the subterranean concrete wall. This leakage is occurring at an active breach that is allowing water to infiltrate on a regular basis. A galvanized steel gutter has been affixed to the wall to collect and shunt the active flow to a drain. Polyvinyl chloride (PVC) piping connects the galvanized gutter to a garden hose that penetrates through the concrete wall at a drilled hole approximately 2 inches in diameter (Figures 69 and 70). Previous attempts to mitigate the leakage at this location are evident. There are indications of water infiltration at the expansion joint in the east subterranean concrete wall near the stair landing leading down to the tram load zone in the south leg of the Arch. The leakage has severely discolored the concrete wall along the edges of the expansion joint (Figure 71). The probable cause of both instances of leakage at the expansion joints is the proximity of the expansion joints to the trench drains located in the ramp leading down to the main exterior entrance to the Arch (Figure 72).



Figure 67. Example of previous repair attempts of cracks in concrete walls. Source: BVH, 2008.



Figure 68. Tongue-and-groove wood slat embedded in concrete wall. Source: BVH, 2008.



Figure 69. Galvanized gutter installed to collect water infiltration at expansion joint in north tram load zone. Note the staining and previous attempts at repairs. Source: BVH, 2008.



Figure 70. Water collection device installed as temporary repair. Source: BVH, 2008.



Figure 71. Water leakage at expansion joint in south tram load zone. Source: BVH, 2008.

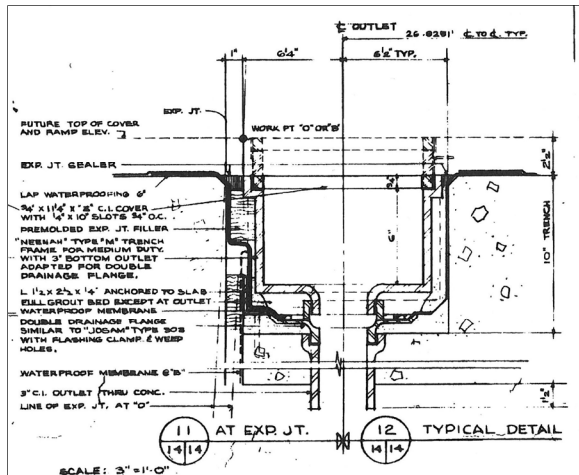


Figure 72. Original construction detail of trench drain located in entry ramps. Note proximity of expansion joint to trench drain. Source: Original Saarinen Construction Documents, detail 11, sheet A14, 1961, JNEM archives, digitized as part of HSR project.



Figure 73. Water infiltration at wall to ceiling junction in north tram load zone. Note accumulation of efflorescence. Source: BVH, 2008.

Water infiltration is also evident in the northeast corner of the upper display area in the north leg of the Arch at the wall to ceiling interface, where accumulations of efflorescence are apparent (Figure 73). This area of leakage is directly below the trench drain located exterior to the structure at the ramp leading to the north entrance of the visitor center. The trench drain is leaking and salts from application of deicing compound appear to have leached into the interior of the lower level of the north leg of the Arch.

The walls in both the north and south legs of the Arch have been sealed with a concrete sealer. This observation was corroborated by JNEM staff. The concrete sealer extends up the walls approximately eight feet above the walking surfaces. The concrete sealer was rolled on with a paint roller and has altered the color of the grey concrete wall, making it darker in appearance than that of the unsealed concrete above (Figure 74). The sealer was reportedly applied by JNEM staff to keep oils from visitors' hands from staining the concrete near handrails and other locations that visitors are likely to touch. Specification information regarding the concrete sealer used was unavailable and therefore the type of sealer applied is not known at this time. Further material research may be required to determine the type of sealer used and date of application.

Staining of the concrete walls was noted at other isolated locations. In the south leg of the Arch on the east wall of the upper exhibit area, streaking of a bituminous/asphaltic material extends down the wall from the ceiling plane approximately 4 feet (Figure 75). Another area of streaking was observed in the north leg of the Arch, on the east wall of the upper exhibit area. This streaking is white in color and extends down from the ceiling plane approximately 3 feet (Figure 76).

Miscellaneous patches and repairs have been made to the walls as part of previous repair campaigns. These repairs range from brushed repairs with an epoxy-like material (Figure 77) to plugging of drilled holes in the concrete wall where handrails were once attached (Figure 78). Due to a lack of routine maintenance records, it is unclear when and what materials were used in the repairs. Further research may be required to ascertain the nature of these repairs and dates of completion.

Various appurtenances such as signage and wall murals are affixed to the concrete walls. Interpretive materials cover much of the concrete wall surface in the display areas and tram load zones. The murals are furred out approximately 2 inches from the plane of the concrete wall. The furring is mechanically attached to the concrete walls with screws and anchors drilled into the concrete walls. The interpretive materials are typically printed on canvas or vinyl and adhered to a plywood backing. Conduit has also been mounted on the wall surface and feeds various items such as the fire alarm strobes, electrical equipment, and a sound system that have been added to the spaces over time (Figure 79).

The tram entrances are formed by block-outs in the concrete wall and are infilled with painted steel panels. The metallic grey paint applied to the panels is in good condition. There are eight such openings in each leg of the Arch. The steel panels are approximately 4 feet 0 inches wide by 6 feet 8 inches high. The tram entrance doors, which are located in the middle of the steel panel and allow access to the tram cabs, measure 2 feet 0 inches wide by 4 feet 6 inches high. In the south leg murals have been placed over much of the west wall of the tram load zone, concealing the concrete wall and the tram load zone doors. In the north leg, the east wall of the tram load zone is exposed concrete (Figures 80 and 81).



Figure 74. Concrete sealer applied to concrete walls. Source: BVH, 2008.



Figure 75. Example of isolated staining on concrete wall in the upper exhibit area of the south tram load zone. Source: BVH, 2008.



Figure 76. Example of isolated staining in upper exhibit area of the north tram load zone. Source: BVH, 2008.



Figure 79. Example of equipment and conduit attached to concrete walls. Source: BVH, 2008.



Figure 77. Example of previous repair campaigns to concrete walls. Source: BVH, 2008.



Figure 78. Patching of concrete wall at former handrail location. Source: BVH, 2008.



Figure 80. Tram entry door and panel, north tram load zone. Source: BVH, 2008 .



Figure 81. Tram entry doors and panels, south tram load zone. Source: BVH, 2008.

The most notable modification to construction as shown on the original documents has occurred in the north tram load zone. Originally designed as a tall, three-story, interconnected space above the tram load zone, portions of the open space were infilled with steel beams and metal deck in 1998 to create additional interpretive and display space on the upper level. This modification reduced clear sight lines to the tram area below. Portions of the partial height wall that separates the walking surface on the upper level from the vaulted space above the lower level and tram queuing lines have been removed and replaced with steel and glass panel guardrails. The condition of the renovated display areas is good, although original material was lost due to these modifications.

The concrete walls in the north and south legs of the Arch were previously fitted with an array of acoustic panels affixed to the wall; it is unclear whether these panels were part of the original construction. The acoustic panels were removed in the 1998 renovation campaign (Refer to 1998 drawings in Appendix L).

A communication console is located in the lower level immediately at the head of the

tram queuing area. The communication console provides communication between JNEM staff stationed at the base of the Arch leg and staff at the observation level. The communication console is comprised of stainless steel panels flush-mounted with the concrete wall and mounted 3 feet 0 inches above the finished floor. The communication console is operable and in good condition.

Doors

Door frames and doors are painted hollow metal. The door frames and doors are in good condition.

Handrails

The original construction drawings indicate all handrails to be 2 inch outside diameter steel pipe handrails, with 1/2 inch brackets installed in steel sleeves embedded in the concrete. The original steel pipe handrails on the stairs leading down to the tram load zones in the north and south legs of the Arch were replaced with stainless steel handrails as part of the 1998 renovation. The replacement handrails meet requirements of the Architectural Barriers Act (ABA) for mounting height and top and bottom rail extensions.

The handrail located at the queuing steps in the south leg of the Arch on the east partial height wall was also replaced in 1998 with a stainless steel handrail that matches the profile of the original pipe railing detailed for this area (Figure 82 and Figure 83). The guardrail on top of the partial height wall at the tram load zone in the south leg of the Arch is original. Composed of painted 5 inch by 2 inch steel tube sections, the guardrail is deficient in height and spacing of voids with respect to the code. The paint on the guardrail is in good condition. A wood slat assembly that mimics a wood fence has been installed to the surface of the partial height wall, presumably to satisfy the code requirements (refer to Figure 83).



Figure 82. Original handrail detail, north tram load zone. Source: BVH, 2008



Figure 83. Reproduction in stainless steel of handrail detail, south tram load zone. Source: BVH, 2008

The queuing railing positioned in the middle of the steps of the south tram load zone is a painted steel pipe railing that was installed in the 1998 renovation. The condition of the paint on the railing is fair as the paint is worn, with primer coat and bare metal exposed.

The handrail located at the queuing steps in the north leg of the Arch on the west partial height wall is painted steel pipe that is original to the design, according to the original construction documents. The original guardrail on top of the partial height wall was replaced in 1998 with an aluminum tube railing and is code compliant with regard to guardrail height and void spacing (Figure 84). The queuing railing positioned in the middle of the loading steps in the north leg is a three legged aluminum railing and is in good condition (Figure 85).

Floors

The floor and stairs of the north and south tram load zones have a terrazzo surface. The terrazzo flooring begins at the entrance to the visitor center and is continuous to the lowest level of the tram load zone, representing a continuity of material through the visitors' spatial progression. The cementitious matrix of the terrazzo surface is creamy-buff in color, with dark and light colored aggregate ranging in diameter from 1/4 to 1 inch in diameter. The terrazzo ranges in thickness from 2-1/2 to 5 inches in depth due to variations in levelness of the slab below. Materials found in the MacDonald archive describe the controversy related to this issue in detail.¹⁰⁶ Silver-colored, zinc-coated dividing strips separate the terrazzo floor segments and range in width from 1/8 to 1/2 inch. The terrazzo flooring was set onto a concrete structural slab.

¹⁰⁶ Letter from Hoel-Steffen Construction to Arthur Girolami Terrazzo Co., January 18, 1968. MacDonald Archive.



Figure 84. Aluminum tube guardrail, north tram load zone. Source: BVH, 2008



Figure 85. Queuing handrail, north tram load zone. Source: BVH, 2008

The terrazzo flooring in the tram load zones is generally in very good condition with minor cracks and hairline cracks evident. Minor patching of the terrazzo floor in the lower levels of the tram load zones is also evident where tram queuing railings have changed configuration (Figure 86). The terrazzo patching has been well executed and blends with the rest of the floor surface. The floor to wall transition is a terrazzo base cove that turns up 4 inches and is flush with the concrete walls. Carpet has been laid over the terrazzo flooring in the upper level interpretive and display areas of the tram load zones with adhesive. The carpet is blue-grey with vinyl transition strips where the carpet abuts the terrazzo. The carpet is turned up at the wall base approximately four inches over the terrazzo base. The top of the carpet that is turned up at the wall is sealed with a sealant material that is different in color from the carpet. The nosings of the stair treads have been painted with slip-resistant grey paint by JNEM staff. The coating is applied annually, according to JNEM maintenance personnel (Figure 87).

A previous study commissioned by the NPS regarding condition analysis and repair recommendations for the terrazzo floor located in the visitor center and lower tram load zones was completed in 2008 by the BVH/WJE team.¹⁰⁷ The study included recommendations for the treatment of terrazzo deterioration, cracking, staining, and general cleaning to return the floor to its original appearance.

¹⁰⁷ NPS Jefferson National Expansion Memorial Gateway Arch; Lobby, Restroom and Terrazzo Repair/Restoration –Final Report Title I, March 2008.



Figure 86. Example of terrazzo patch at queuing handrail support, south tram load zone. Source: BVH, 2008



Figure 87. Slip-resistant paint applied to nosing of terrazzo stairs. Source: BVH, 2008



Figure 88. Example of worn paint surfaces, tram load zone. Source: BVH, 2008

Ceilings

The ceiling plane of the north and south legs in the tram load zone is cast-in-place concrete. Spray-on acoustic material has been applied to the ceiling plane and painted black. Both the spray-on acoustic material and the paint are in fair condition. The black coating turns down the wall to approximately 12 inches below the wall/ceiling interface. Various conduits and drains are run along the ceiling plane and are painted black as well. In the upper level of the tram load zone, the ceiling is suspended acoustical tile, which is in fair condition.

Observation Level

Walls

Tram Load Zones

The walls of the observation level are composed of the triangular steel sections that make up the structure of the Arch, with the inner skin of the structural section used as the finish substrate of the observation level. The walls immediately adjacent to the tram exits are painted steel. The paint is metallic grey in color and is in poor condition. Graffiti has been scratched into the painted surface in various locations and additional areas of paint are worn away (Figure 88). Attempts have been made to paint over scratches, graffiti, and worn areas of the painted surface. The walls opposite the tram exits are covered with heavy duty carpet similar to the carpet used on the walls of the observation level. The wall carpet is stained with dirt at the supply registers (Figure 89). Toward the top of the exiting area are removable hollow metal partition walls that allow access to the trams and tram equipment beyond.



Figure 89. Wall carpet applied to outer steel skin at upper tram load zone. Note soot stain at supply register. Source: BVH, 2008



Figure 90. Observation level viewing ports. Note original aluminum oxide granulated finish on foot rest, as shown in the lower portion of this figure. Source: BVH, 2008

Observation Level

The walls of the observation level are comprised of the steel sections that make up the structure of the Arch. According to the original construction documents, the inner skin of the structural section and the bracing are comprised of 3/16 inch steel plate and were specified to receive a factory applied vinyl plastic finish.¹⁰⁸ Currently, the walls are covered with a heavy duty carpet material that is glued to the steel panels with adhesive. The carpet material used by JNEM staff is Tretford wall carpet manufactured by Eurotex. The carpet material is currently in fair condition. The wall covering is routinely changed or patched due to the wear that it receives. The wall covering is difficult to maintain, according to JNEM staff, as the adhesive is prone to failure. Various patches are evident in the carpet material and the patches are of different dye lots. A vinyl base covers the edges of the wall carpet at the bottom, while the top is an exposed cut edge held away from the wall ceiling transition approximately 2 inches (Figures 90 and 91). The lower portion of the observation walls consists of an extended sloping platform formed from 1/4 inch steel plate, against which visitors can lean while looking through the window ports. The platform is covered with the same carpet material as the rest of the wall. The underside of the platform consists of a concealed cove light that washes the lower portion of the wall and the foot rest (Figure 92). The lower half of the observation level wall was specified to receive the same aluminum oxide granulated finish as the floor and has subsequently been carpeted over. The foot rest has the original aluminum oxide finish intact and exposed (refer to Figure 91).

¹⁰⁸ Letter to MacDonald Construction Company from Bruce Detmers of Eero Saarinen and Associates, April 7, 1964.



Figure 91. Wall carpet at triangular bulkhead, observation level. Source: BVH, 2008



Figure 92. Cove lighting at underside of observation level viewing wall. Source: BVH, 2008



Figure 93. View of north tram load zone stairs from observation level. The control booth with communication console is located center-right in this photograph. Source: BVH, 2008

The north and south walls of the observation level contain control booths that house communication and tram control consoles (Figure 93). The consoles are similar to the communication and control consoles found in the tram load zone at the base of the Arch. The consoles are fabricated from stainless steel and are in good condition. The control booths are open to the observation level. The walls are clad with the same carpet material as the walls of the observation level and are in fair condition. A folding seat is located on the wall opposite the communication equipment.

Fire alarm strobes are also located on the north and south walls of the observation level. Directional signage is mounted on both the north and south walls of the observation level indicating tram designations “NORTH TRAM” or “SOUTH TRAM.” The placards are approximately 4 inches wide by 12 inches long with white lettering on a black background, and are covered with clear plexiglass.

Emergency Lavatory

An emergency lavatory is located in the south wall of the observation level and was designed and installed per the original construction documents. The lavatory contains a chemical toilet for emergency use. The emergency lavatory is separated from the observation level with a plastic, faux-wood accordion style partition.

Windows

The observation level has thirty-two window ports, each approximately 7 inches high by 24 inches wide, with sixteen on the east and sixteen on the west, from which visitors can view the vistas of the Mississippi River and the urban surroundings of the park.

Observation windows are composed of 3/4 inch laminated glass set into a stainless steel frames (Figure 94). According to JNEM staff,

the glazing is routinely replaced due to scratching and graffiti etched onto the surface of the glass. The windows are in good condition, although some original parts such as screws and pressure lock hasps have been replaced over time, according to JNEM staff.

Floors

Upper Tram Load Zone

The stairs of the load zone consist of 1/4 inch thick bent plate steel sections supported on steel supports welded to the wall of the Arch structure. The stairs immediately adjacent the tram exiting zone varies in riser height and tread width. At the lowest portion of the run the stairs have 8 inch risers and 9 inch treads, while at the upper portion of the tram load zone the stairs have 5 inch risers and 12 inch treads. The treads of the exiting zone are 1/4 inch thick steel plate with an aluminum oxide granulated finish (abrasive steel), with aluminum nosing painted black (Figure 95). Yellow paint has been applied to the treads to signify the immediate tram loading area. Both the black and yellow paint applied to the plate metal are in fair condition.

Observation Level

The floor of the observation level is composed of removable 1/4 inch abrasive steel plate. The floor of the observation level is carpeted with the same carpet that is found on the walls. The carpet is in fair condition. Replacement of sections of the floor carpet is evident, with carpet sections from differing dye lots used (Figure 96). The floor surface follows the curve of the Arch. An access hatch, located at the centerline of the Arch, provides access to the tram electrical equipment housed in the interstitial space below the observation floor. The hatch is covered with the same carpeting as used on the rest of the floor surface (Figure 97).



Figure 94. View port window in open position.
Source: BVH, 2008

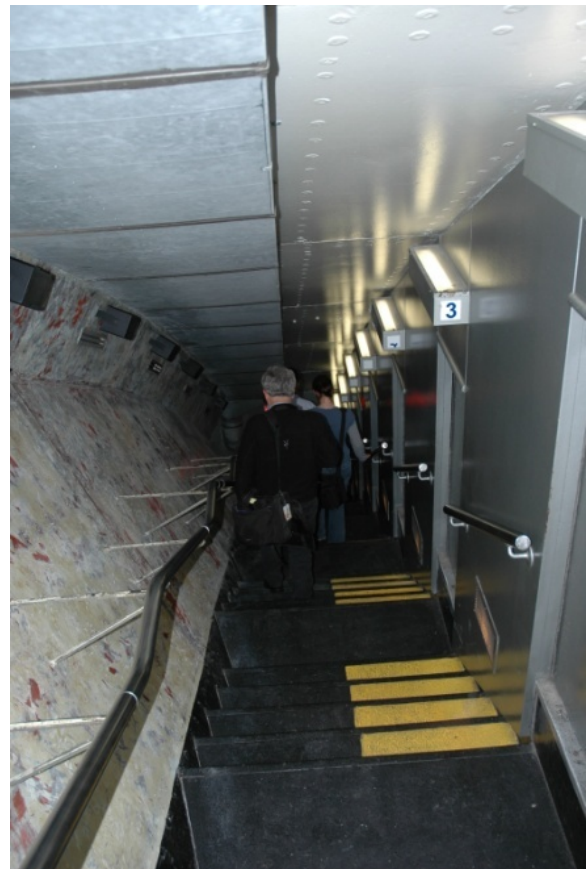


Figure 95. View of north tram load zone. Source: BVH, 2008



Figure 96. Floor carpet located at the observation level. Note color variation of carpet. Source: BVH, 2008



Figure 97. Floor access hatch to tram electrical equipment, observation level. Source: BVH, 2008



Figure 98. Tram load zone handrails. Source: BVH, 2008



Figure 99. Access hatch to top of Arch in open position. Source: BVH, 2008



Figure 100. Tram electrical equipment located below observation level floor. Source: BVH, 2008

Control Booth

The floors of the control booths are composed of the same 1/4 inch abrasive steel plate that makes up the floor of the observation level and are clad with the same carpet material. The floor is flat and not curved like the observation level floor.

Handrails

There are handrails on both sides of the exiting zone (Figure 89). The handrail on the outboard side extends approximately 24 inches from the triangular section of the Arch. This handrail is composed of 1/2 inch bent steel bars with a 2 inch outside diameter stainless steel pipe handrail, and is original to the Arch construction. The handrail is unfinished and is in good condition. The inboard handrail extends approximately 3-1/2 inches from the wall and is composed of 2 inch outside diameter stainless steel pipe. This handrail is unfinished and is in good condition.

Ceilings

The ceiling of the observation level, tram load zone, control booths, and lavatory is composed of the inner skin of the steel sections that comprise the structure of the Arch. Steel strapping covers the Arch sections in the observation level, creating a grid, and is painted white. The paint finish is in good condition. An access hatch to the top of the Arch is located at the apex of the Arch at section line 0. The access hatch is round and is approximately 24 inches in diameter and is in good working order (Figure 99). A rectangular steel plate covers the access hatch to blend with the rest of the ceiling plane.

The ceiling of the tram load zone has been retro-fitted with an additional 30 inch by 6 inch galvanized supply air duct fit tight to the existing ceiling plane.

Equipment Space

The space below the observation level, from the underside of the floor material to the intrados of the Arch's triangular section, contains electrical and tram equipment (Figure 100). The walls are the inner skin of the triangular section of the Arch and are coated with primer only. The primer is in good condition. The equipment space is accessed by means of an access hatch in the observation level floor and a ladder extends down to the intrados of the triangular section.

Tram Capsules

The tram capsules are the mode of conveyance to the top of the Arch and back down. The capsules are circular drums approximately 5 feet 0 inches in diameter and 6 feet 1 inch deep at the centerline. The capsules are comprised of 0.09 inch thick aluminum panels on a 1 inch square aluminum tube frame. The interior of the aluminum skin was originally specified to receive a factory applied vinyl plastic finish. As part of annual maintenance procedures, JNEM staff routinely repaints the interior of the capsules. The interior of each capsule contains five molded plastic seats designed by Saarinen (Figure 101), with eight capsules per leg of the Arch. The entrance to each capsule is composed of two center opening doors, each approximately 1 foot 1 inch wide, with two 6 inch wide by 13 inch high glazed openings per door that are filled with 1/4 inch thick clear plastic glazing. The doors were also specified to receive a factory applied vinyl plastic finish. The floor of the capsule is composed of 3/16 inch aluminum with an aluminum oxide granulated finish that extends to the underside of the molded plastic seats. All of the components of the capsules are in good condition, as JNEM staff routinely maintains the capsules. See Appendix B for further information regarding paint analysis conducted on the trams.

Legs of the Arch

The legs of the Arch are accessed from the top of the Arch via a hollow metal door located at the bottom of the tram load zone and signed as “EMERGENCY EXIT ONLY” (Figure 102). The legs of the Arch are composed of triangular steel sections with the inner skin of the sections exposed to view. The steel panel walls, structural steel, steel supports, and steel bracing located in the legs of the Arch were



Figure 101. Interior of tram capsule showing Saarinen-designed seats. Source: BVH, 2008



Figure 102. Emergency exit located at the bottom of the tram load zone. Source: BVH, 2008

specified to be factory primed with red lead primer and painted with a field applied grey top coat. The legs of the Arch are essentially a large void space with a series of spiral stairs and switch-back stairs and landings composed of 1/4 inch “checkerplate” with medium pattern. The stairs and landings are in fair condition, with corrosion on some stairs and landings (Figure 103). Handrails are 2 inch outside diameter steel pipe primed and painted, with a grey topcoat. The handrails are in good condition. Various sections of the stairs are enclosed with fencing material composed of 2x2 welded wire fabric (Figure 104). Refer to the Structural section, below, for a discussion of the exposed interior structure of the Arch.

Tram Maintenance Bay

A maintenance bay is located in the base of each of the Arch’s concrete foundations. The bays provide access for JNEM staff to the tram capsules for routine maintenance of the capsules. The walls of the maintenance bays are smooth finished concrete painted yellow. The floors are smooth finished, unsealed grey concrete (Figure 105). The maintenance bays contain work benches and tool storage. Concrete steps lead down to the lower queuing area of the tram load zone. The maintenance bays are in fair condition.



Figure 103. Corrosion on checkerplate landing, interior of Arch leg. Source: BVH, 2008



Figure 104. View of interior of Arch leg. Source: BVH, 2008



Figure 105. View of the tram maintenance bay. Source: BVH, 2008

Structure

Review of Construction

The Gateway Arch was constructed in segments in much the same way as modern long-span cable stayed bridges are, with a completed segment of specified length and geometric shape brought to the construction site either partially or wholly assembled, ready to be fastened to the existing structure. The triangular tube segments ranged from 12 feet in height at the base to an 8 foot tall keystone segment at the top. Each of the segments was set into place with either conventional ground-supported cranes for the segments within the first 72 feet above grade or creeper derrick cranes attached to the legs of the constructed portion of the Arch when construction had surpassed 72 feet.

Each evening after a section of the Arch was placed; the section location was surveyed using a theodolite scope and triangulation of the readings. Upon completion of surveying, readings were reviewed and calculations completed to determine if any changes to the set of an Arch section were required prior to welding. Positions were surveyed at night when there were no movement effects due to solar radiation and in order to assure that the legs would be in alignment when the top was reached.¹⁰⁹ The surveying was also completed at night because the temperature was more constant, thus limiting displacements caused by temperature differences (Figure 106).¹¹⁰

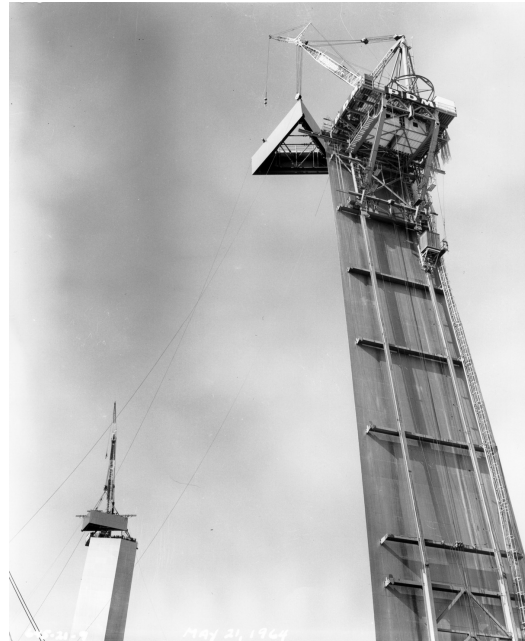


Figure 106. View of initial placement of a segment of the Arch completed from the creeper derrick crane. Source: JNEM archives, image V106-4048.



Figure 107. Shop assembly of the Arch segments, showing internal temporary framing for placement and installation. Source: JNEM archives, image V106-3946.

¹⁰⁹ Kenneth J. Kolkmeier, "Layout and Erection Control of the St. Louis Memorial Arch," presented at the ASCE Annual Meeting and Environmental Engineering Conference, October 12–22, 1965.

¹¹⁰ George B. Hartzog, Jr., Director of the National Park Service, 1964–1972, "Will the Arch Stand?" in Moore, *Gateway Arch: An Architectural Dream*.

All of the vertical and horizontal shop welds on the exterior between plates were completed as single pass welds to create a smooth and uniform appearance. Mr. Kolkmeier provided insight about the welding process and stated that all welds were performed to American Society of Mechanical Engineers (ASME) standards and were X-rayed. Most stainless steel welds were performed in the shop, except as implemented for joining sections in the field. All exterior welds were argon gas/CO₂ shielded. All interior welds were hand welded, while exterior welds were performed using a machine/jig.¹¹¹

As completed the field welds are not as neatly done as the shop welds. Oftentimes the field welds were completed as multi-pass welds in order to accommodate tolerances between the triangular tubular sections. The field welds on the exterior are multi-pass full penetration groove welds, with the reinforcement above the plate not ground smooth. The multi-pass welds vary in size as necessary to accommodate dimensional variances. Some of the horizontal field welds between successive sections appear to be discolored, which is most likely attributed to atmospheric soiling (Figures 108 and 109). Mr. Kolkmeier indicated that the field welds at the stainless steel plates on the exterior were not ground flush as an architectural decision, and helped establish the pattern on the skin of the Arch that was desired by Saarinen.¹¹² The locations of the plug welds where the creeper derricks were attached were ground flush and are thus undetectable from grade.



Figure 108. View showing discoloration of field welds near the base of the Arch. Source: WJE, 2008.



Figure 109. View showing discoloration of field welds near the base of the Arch. Source: WJE, 2008.



Figure 110. Typical segment above concrete fill (Station 0 to Station 44) constructed as three-L-sections. Source: Ken Kolkmeier.

¹¹¹ Kolkmeier, interview by Worth et al., January 14, 2009.

¹¹² Ibid.



Figure 111. View showing excavation for concrete footings of Arch. Source: JNEM archives, image V106-3865.



Figure 112. View showing completed reinforced concrete footing. Source: JNEM archives, image V106-3938.

The lowest four tube segments (Stations 68, 69, 70 and 71) for both legs were entirely shop assembled as singular large triangular segments, shipped to the site, and installed. Thus, the only field welding required at these sections was between triangular sections at the horizontal station lines.¹¹³ The remaining segments up to the 300 foot level (Station 45) were fabricated in the shop as three rectangular panels (one for each side of the triangular section).

As part of the on-site assembly, the corners were field welded together joining the three sections into one triangular tube section, and pick point plates were installed at the intrados corners for creeper crane lift cables. By welding pick points to the segments, the cables could be adjusted in length to make fine adjustments, assisting in fitting the section into place. When the final location was determined by the surveying process described above, the segments were field welded at the station joint between segments. The segments above the concrete fill (Station 44 to Station 0) were constructed as three L-sections, so the field welding occurred within the plates rather than at the corners. Again, these segments were assembled on site, hoisted into place, fitted, and welded to the segments below.

The footing excavation for each leg of the Arch was 75 feet wide by 94 feet long, extending to a depth of nearly 50 feet at the southeast portion of the south leg corner in order to reach bedrock (Figure 111). The concrete footing has four steps, each approximately 10 feet deep (Figure 112). The initial concrete placement for the south leg foundation consisted of 2,400 cubic yards of

¹¹³ Kolkmeier, interview by Worth et al., January 14, 2009.

concrete and took nearly twenty-three hours to place.¹¹⁴

Concrete was placed continuously in order to avoid a cold joint in the foundation pad that could later present structural inadequacies. Formwork was erected to create a triangular void within each foundation leg in order to provide the required space for the tram load zone and elevator pit, as shown in Section 1 and Section at Elevator Pit, both on drawing sheet S102 in Appendix I. The remaining portion foundation pads of each leg were constructed in seven pours, with each pour having a depth of 5 feet. Each placement of concrete consisted of two 2-1/2 foot lifts of concrete with retarders used in the first, lower mix to prevent a cold joint between the 2-1/2 foot lifts.¹¹⁵ The pours were keyed together with a series of grooves. A series of the post-tensioning bars (1-1/4 inch diameter) were installed and anchored at 34 feet and 24 feet below grade. These two levels of post-tensioning bars were offset to ensure that the entire tension load of the bars was not concentrated at one location within the footings (Refer to “Section 2” on sheet S102 in Appendix I). Two hundred and fifty-two post-tensioning bars were placed in each leg (126 bars in each of the two outside/extrados corners) and continued to the 300 foot level where the reinforced concrete fill terminates, as shown in Figures 114 and 115.

From foundation level to the 300 foot level (Station 45), the interstitial three foot space between the inner and outer plates of both Arch legs was filled with reinforced post-tensioned concrete (Figure 116).

¹¹⁴ Ted Rennison, “Laying the Foundations,” in Moore, *Gateway Arch: An Architectural Dream*.

¹¹⁵ Memorandum from B. A. Prichard of MacDonald Construction Company, St. Louis, Missouri, to Bruce Detmers of Eero Saarinen and Associates, June 4, 1962, Ref. Contract No. 14-10-0232-462.



Figure 113. View construction site with completed concrete foundation in foreground. Source: JNEM archives, image V106-3897.



Figure 114. Post-tensioning bars in extrados corners at the concrete footings. Source: JNEM archives, image V106-3926.



Figure 115. Overall view in archival photograph, showing concrete footing and post-tensioning bars in extrados corners. Source: JNEM archives, image V106-3877.

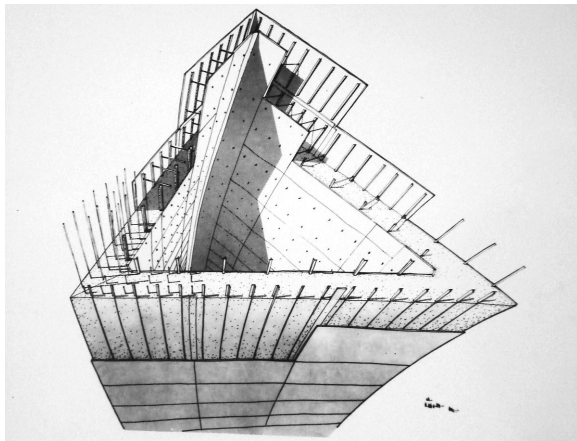


Figure 116. Schematic drawing, showing composite section of the Arch below the 300 foot elevation. Source: Saarinen archives, Yale University, New Haven, Connecticut.

Steel stiffener angles are fastened to the interior plate of each section with 3/8 inch diameter stud bolts, to which are attached additional stiffener angles welded to the exterior plate. The stiffener angles, stud bolts and post-tensioning strands work with the prestressed concrete to create a composite section,¹¹⁶ in which the concrete and the steel skin plates create a structural member that act as a single unit resulting system has greater load carrying capacities than the sum of its parts. Refer to drawings S103, S104, and S107 in Appendix I.

During the construction of the Arch, an inward thrusting force caused by the leaning of each leg was present, which would typically be carried by the keystone unit of an arch when in place. This inward thrusting causes significant tensile stresses within the individual segments of the arch legs, which are only experienced during construction, because the structure of an arch is not resolved until the keystone is place. A completed arch, in theory at least, provides a structure that eliminates tensile stresses, as all the forces are resolved into compressive stresses. This is useful because concrete can strongly resist compression but is very weak when tension, shear or torsional stress is applied. By using the arch configuration, significant spans can be achieved. This is because all the compressive forces hold it together in a state of equilibrium.

While the arch is an effective structure, the challenge was to provide an alternative force mechanism to keep the sides of the arch from falling inward before the keystone was placed. Consequently, the Arch position needed to rely on either large tieback cables or another

¹¹⁶ Composite section: A structural member composed of two or more dissimilar materials joined together to act as a unit in which the resulting system is stronger than the sum of its parts.

mechanism to hold the inward deflection of the Arch to within the specified engineering tolerances. A composite member consisting of prestressed concrete fill reinforced with post-tensioning reinforcing bars along with the inner and outer skins was designed to resist the gravity loads causing inward deflection of the Arch legs and the overturning moment, thus eliminating the need for large tieback cables. The reason for using this composite section was to achieve a structural member consisting of the “steel” cladding and prestressed concrete fill that better utilized the materials strength and stiffness, benefiting from the compressive strength of the concrete along with the tensile strength of the steel (exterior and interior plates, as well as post-tensioning rods).

Prestressed concrete is a design method for overcoming the natural weakness of concrete in relation to tensile forces. It is often used to produce structural members that have longer spans and require greater capacity than is practical with typical reinforced concrete. Prestressing tendons (bars) are used to provide a clamping load, which produces a compressive stress that offsets the tensile stress experienced by a concrete member due to a typical bending load (in this case the self-weight of the legs and wind forces). Prestressing can be applied to concrete members in two ways, by pretensioning or post-tensioning. Prestressing by post-tensioning involves installing and stressing bar tendons after the concrete has been placed, hardened and attained a minimum compressive strength for that transfer.

The post-tensioning bars and temperature reinforcing steel were integrated within the concrete filled portion of the Arch to induce compressive forces in the concrete and increase strength by tensioning the steel bars, to effectively carry the design loads during construction (Figure 117). The 1-1/4 inch

diameter post-tensioning steel bars and sleeves were held in place by steel positioning plates with holes drilled to the appropriate bar spacing. The bars needed to be inclined in two directions in order to follow the double curvature of the Arch. The fitting of the 126 bars in the cross section at the base was not difficult, but became more congested as the size of the cross section decreased with the taper of the tubular leg sections. Each bar was tensioned to 142,000 lbs (approximately 115 ksi bar stress), continuing to the top of the concrete fill. Post-tensioning occurred for each of the bars when the concrete fill reached a compressive strength of 4,000 psi. The required compressive strength was typically reached in seven to ten days after placement and was approximately 80 percent of the design strength. The concrete was placed in approximately 5 foot lifts and terminated 1 foot short of the segment height, to permit installation of the steel positioning plates. Tensioning of the bars was performed at the locations shown on drawings S111 and S112 when the concrete reached the required compressive strength. The 142,000 lb stressing for each bar was done by a hydraulic jack, with a total of 18,000 tons of prestress applied per leg. The design engineers specified tensioning the bars in sequence to balance the loading on the existing structure and so as not to overstress the concrete fill. (Information on the specific sequence for loading was not identified during archival research for this study.) The full tension load was applied to each bar in one operation with a 100 ton capacity hydraulic jack, which reacted against a steel jacking plate embedded at the top of the concrete (Figures 118 and 119).¹¹⁷

¹¹⁷ Joe Jensen, “Facts about the Construction,” in Moore, *Gateway Arch: An Architectural Dream*.



Figure 117. Overall view showing integration of the reinforcing bars at the concrete footing and concrete filled portion of the steel skins. Source: JNEM archives, image V106-3940.



Figure 118. View of tensioning the reinforcing bars below the 300 foot level. Source: JNEM archives, image V106-3933.



Figure 119. View of post-tensioning of the reinforcing bars below the 300 foot level. Source: JNEM archives, image V106-3974.

The general concrete reinforcing layout inclusive of post-tensioning steel is shown in drawings S109, S111, and S112 in Appendix I. The closure reinforcing details at the termination of the concrete fill at Station 45 are shown in S108 also in Appendix I.

Above Station 45, the inner and outer skins are connected together using a series of carbon steel stiffener angles, diaphragms, 1/2 inch diameter bars, and bent steel plates in a cellular type of construction, similar to aircraft design. The steel stiffener angles were spaced based on a ratio of the panel and tube width.¹¹⁸ The stiffener angles (2 inch by 2 inch by 1/4

¹¹⁸ The stiffener angles are horizontally spaced at two-fifths of the panel width, $2V/5$ in feet at the interior skin plate and one-fifth of the panel width $V/5$ in feet at the inside face of the exterior skin plate. This dimension is clarified by the structural drawings S107 and S108 included in Appendix I. Ten panels exist at each face of the triangular tube section. The panel width, V , is equal $W/5$; W is equal to half the dimension of one of the exterior faces of the tube. For example at the base the exterior face of the tube is equal to 30 feet, thus $W=15$ ft, and the panel width, $V=3$ feet, and the spacing of the stiffener angles is 1.2 feet or $2/5 \times 3$ feet at the interior skin plate or 0.6 feet at the exterior skin plate.

inch) are stitch welded to the back side of the exterior stainless plates with fillet welds. A welded built-up stiffener angle, fabricated from a 2-inch by 1/2-inch steel plate and a 1/4-inch steel plate of width equal to the space between interior and exterior skins, is bolted to the interior carbon steel plates (refer to Section 3 at Corners on drawing S113 in Appendix I). The interior and exterior skins are further tied together with diagonal rod braces. Further description of bolt sizing and spacing is included as part of the Condition Assessment, below.

A secondary measure to provide stability against the inward acting bending moments on the cantilevered Arch legs was implemented near the top of the Arch. When the legs reached an elevation of 530 feet, about 100 feet from the top of the Arch, a large trussed strut was installed between the legs. This additional measure of construction stability was deemed necessary by the contractor to ensure the stability of the cantilever legs, while simultaneously limiting the stresses on the post-tensioned concrete. The trussed strut is shown in Figures 120 through 122.



Figure 120. Close up view of the trussed strut installed during construction to resist overturning and deflection of the cantilevered Arch legs. Source: JNEM archives, image V106-4111.



Figure 121. View of the trussed strut installed during construction to resist overturning and deflection of the cantilevered Arch legs. Source: JNEM archives, image V106-4119.



Figure 122. Overall view of the strut installed during construction. Source: JNEM archives, image V106-4124.

The Pittsburgh-Des Moines Steel Company (PDM) helped with developing the construction sequencing and project work plan. PDM had an internal engineering design group that was involved in developing structures for nuclear power plants, buildings, and bridges around the country.

Initially, cables and guy wires were included in the plans for stabilizing the legs during construction, as shown in S120 in Appendix I. Utilizing cable tie-offs would require independent cranes, and the initial approach was to use two 600-foot-tall derricks to build the Arch. Mr. Kolkmeier indicated that with this initial approach, “guy lines would have had to been run out into the river to support the derricks but the Corps of Engineers objected”; thus the concept of using creeper cranes evolved, and with it came a new concept for stabilizing the legs of the arch during construction. The use of a strut, originally envisioned as a hinged strut, at the 300 foot level to stabilize the two legs of the Arch was an idea conceived by PDM.¹¹⁹ The design was modified for the strut to be

¹¹⁹ Kolkmeier, interview with Worth et al., January 14, 2009.

installed when the legs reached an elevation of 530 feet, at Station 22. The design and dimensions, along with the connection detail of the trussed strut to the completed portion of the Arch legs, are shown in the shop drawings for the strut prepared by Richardson, Gordon & Associates, drawing numbers 441-29, 441-30, and 441-31 (refer to Appendix K). Hydraulic jacking conducted while placing the strut helped the engineers to determine the closure pressure at the top of the Arch when the last section was placed. The calculated pressure was ultimately found to be within 5 percent of the actual pressure. Upon completion of setting the keystone segment, the strut was taken out, which gave the engineers an opportunity to adjust the closure—similar to closures in bridge structures.¹²⁰

Previous Structural Studies

The following proceedings and reports of previous structural studies were reviewed as part of the condition assessment relating to the structural aspects of the Gateway Arch. A summary of the findings and recommendations presented in each previous structural study is provided below.

- Report by D.B. Steinman, Consulting Engineer, New York, *Jefferson Memorial Arch—Aerodynamic Studies*, December 31, 1948. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964, but was not available for review during this study.

¹²⁰ The closure element is a structural member that joins two or more structural elements (tube sections—keystone segment to adjoining Arch segments), where tolerance discrepancies cannot be accounted for in the individual elements. This closure piece provides continuity to create one arched element rather than two curved cantilevered columns. The closure allows the loads to be redistributed as compressive forces within the arch.

- Report by D.B. Steinman, Consulting Engineer, New York, *Jefferson Memorial Arch—Supplementary Aerodynamic Studies*, February 24, 1949. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964, but was not available for review during this study.
- Report No. 314-001 by Fairchild Aircraft and Missiles Division, *Jefferson National Expansion Memorial Arch—Dynamic Analysis*, April 27, 1960. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964, but was not available for review during this study.
- Report No. 314-002 by Fairchild Stratos Corporation, *Jefferson National Expansion Memorial Arch—Dynamic Analysis*, October 23, 1961. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964, but was not available for review during this study.
- Undated report in German, entitled *Sicherheitsnachweis für seitliches Ausknicken des Bogens [Safety Certification of the Arch against Sideways Buckling]*, by Dr. Konrad Sattler, Technical University, Berlin. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964, but was not available for review during this study.
- Severud-Elstad-Krueger Associates, *Data Book 3220* (subsequent computations received March and April 1964). This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964, but was not available for review during this study.
- *Report on the Jefferson National Expansion Memorial Gateway Arch* by Edwin Rose (Chief, Structural and Architectural Branch) and Harvey C. Olander (Head, Bridges Section), U.S. Department of Interior—Bureau of Reclamation, Division of Design, Office of Chief Engineer, Denver, Colorado, December 1964.
- *Official Report of Proceedings before the U.S. Department of Interior - Bureau of Reclamation on the Structural Aspects of the Gateway Arch, St. Louis MO*, February 3, 1965.
- *Aerodynamic Stability of the Jefferson National Expansion Memorial Gateway Arch*, by Lloyd R. Cayes and Charles F. Scheffey of the Structures and Applied Mechanics Division of the Office of Research and Development in the U.S. Department of Commerce Bureau of Public Roads, in collaboration with George S. Vincent, Consultant to the Office of the Secretary of the U.S. Department of the Interior, July 1965.
 - “Appendix A—Field Study of Partially Completed Arch,” by Robert F. Varney, April 1965.
- *St. Louis Entrance Blast Vulnerability Assessment and Conceptual Retrofit*, prepared for the U.S. Army Corps of Engineers, Omaha, Nebraska, by James W. Wensevich, Kelly Thomas, J. Hui Geng, and Charles J. Oswald, March 25, 2004.
- *Vulnerability Assessment: Jefferson National Expansion Memorial St. Louis, Missouri*, prepared for the National Park Service Jefferson National Expansion Memorial by Black and Veatch Special Projects Corps, February 13, 2003.
- *Blast Assessment and HVAC Study for the Jefferson National Expansion Memorial St. Louis, Missouri*, prepared for the National Park Service, Denver Service Center by Leo A. Daly and Cermak, Peterka, and Petersen (CPP), December 14, 2007.

Report on the Jefferson National Expansion Memorial Gateway Arch Department of Interior—Bureau of Reclamation, Division of Design, Office of Chief Engineer, December 1964

A copy of this report is included in Appendix C (without appendices). In a letter dated February 10, 1964, from the Acting Commissioner of the Bureau of Reclamation, the Bureau's Chief Engineer was asked to provide technical assistance to the National Park Service regarding how to resolve the complex structural design of the Gateway Arch. As a result of this request, an independent review of the structural design was conducted. This report presents results of the independent review and analysis of the structural design of the Gateway Arch based on the construction documents (inclusive of drawings, specifications, data, etc.) made available by the National Park Service, Eero Saarinen and Associates, and Severud Elstad Krueger Associates. Review of the provided material raised questions about assumptions and data, specifically relating to the aerodynamic stability of the Arch. The aerodynamic stability was investigated utilizing wind tunnel tests of a scale model. The results and conclusions of that test are outlined in the *Aerodynamic Stability of the Jefferson National Expansion Memorial Gateway Arch*, July 1965.¹²¹

The design criteria for the structural review were established by assessment of supporting studies, historical records, and climate data. Special studies were conducted to determine

magnitude of wind and seismic loads, as well as thermal gradient. The Arch was already constructed to a height greater than 160 feet when the Bureau's independent study was requested. The Bureau indicated that, had its involvement occurred sooner, it would have conducted wind tunnel tests on a model of the final design of the arch, scaled for mass and stiffness, to verify aerodynamic stability. The Bureau also suggested that research should have been conducted on the properties of the construction materials prior to beginning construction to understand their thermal characteristics. The Bureau also recommended a program of instrumentation to measure temperatures and strains at critical points in the arch construction. An interim report submitted by the Bureau on June 1, 1964, found structural deficiencies at details above Station 45; recommendations for corrections were made, as described later in the report. A meeting of the consultants, the NPS, and the Bureau was held in reference to the deficiencies. The consultants presented proposed changes to the design including a shorter gap in the longitudinal diaphragms and an additional strut at each of the transverse triangular frames. The NPS requested that the Bureau recheck the modified design and provide further information about the study of metal temperature.

Upon review of the design data inclusive of calculations, specifications, drawings, and previous structural studies and reports, the Bureau indicated that the following specific weaknesses in the structure had not been adequately addressed.

- No studies were made regarding the effects of stopping the longitudinal diaphragms between the exterior and interior skin plates.
- No studies were made to determine a rational basis for analysis of the rigid-type corners of the transverse triangular frames.

¹²¹ Lloyd R. Cayes and Charles F. Scheffey of the Structures and Applied Mechanics Division of the Office of Research and Development in the U.S. Department of Commerce Bureau of Public Roads, in collaboration with George S. Vincent, Consultant to the Office of the Secretary of the U.S. Department of the Interior, *Aerodynamic Stability of the Jefferson National Expansion Memorial Gateway Arch* (July 1965).

- Studies regarding aerostatic wind forces were not made.
- Thermal studies were not as thorough as required.
- The physical and thermal properties of stainless steel were not and should have been thoroughly investigated.
- The stability of the Arch as a single structure was not evaluated.
- Earthquake loadings should be considered in the design for the Arch.
- The Arch should be able to withstand an earthquake of intensities ranging from VI to VII, Modified Mercalli Rating.

Upon review of the design data inclusive of calculations, specifications, drawings, previous structural studies and reports, the Bureau outlined the following inadequacies or undesirable details in the original design above Station 45:

- Discontinuity in the longitudinal diaphragms caused high stress concentrations in the skin plates near the ends of the diaphragms.
- The rigid corner detail of the transverse triangular frames caused severe local stresses, creating elastic instability conditions. Transverse thermal stresses caused by this detail also contribute to the severe local stresses.
- The high thermal coefficient of expansion of stainless steel contributed to high thermal stresses. The thermal coefficients of carbon steel and stainless steel vary significantly, indicating that with the same temperature differential, the stainless steel will expand and contract differently than the carbon steel, possibly inducing stress into the structure.
- The use of double spot fillet welds as compared to continuous fillet welds originally specified for welding the stainless steel exterior plate to the longitudinal diaphragms. The welds are at critical areas in the cellular structural

element; this condition is enhanced when the diaphragms are discontinuous.

Upon review of the design data inclusive of calculations, specifications, drawings, previous structural studies and reports, the Bureau outlined the following inadequacies or undesirable details in original design between Stations 45 and 71:

- Stud welds and double spot fillet welds for connecting the stainless steel plate to the stiffeners to develop composite action were severely overstressed and could shear or separate from the concrete due to creep, shrinkage, and temperature change, thus rendering the stainless steel exterior plate ineffective as part of the composite section. This welding technique can also create undesirable effects on the appearance of the Arch.
- Other deficiencies may exist, but because the lower portion of the Arch had already been constructed these were difficult to verify and or correct. Strengthening of the completed portion of the Arch could only be done by adding structural members, thus a study was conducted as part of this analysis, rendering the exterior plate ineffective and the concrete cracked. The stresses on the interior carbon steel plate were high, but not excessive.

Preferred Recommendations

Based on the aforementioned deficiencies in the structural design at the time of this review, the following conclusions were made and repairs recommended as part of the study:

- The deficiencies in the structural design above Station 45 required drastic changes and sections completed above Station 45 needed to be removed and refabricated to change the exterior skin plate. The Bureau's recommendation included a radical change in design, abandoning the cellular-type of construction for a more typical skeletal frame design with stainless

steel cladding. The proposed redesign of the Arch Section above Station 45 is shown in Chapter 5, Arch Detail Analysis of the “Report on the Jefferson National Expansion Memorial Gateway Arch,” Drawing No. X-0A-D913.¹²² Refer to Appendix L for a copy of this drawing.

- The proposed design would limit and nearly eliminate thermal stresses, which were the most severe stresses.
- The proposed design would limit some of the problems associated with the double spot welding technique.
- Little or no cost increase to the amount of material would be required, with a reduced cost of erection.

Alternative Recommendations

The following recommendations were made by the Bureau in order to achieve a structure with safety factors greater than 1.0, if the proposed recommendation of the stainless steel cladding option was not selected to repair the section of the Arch above Station 45.

- The longitudinal diaphragms were to be made continuous. Proposed repair details were included as part of the study
- The corner detail of the transverse triangular frame needed to be changed to provide a positive hinged connection.
- The distance between the interior and exterior skins between Stations 27 and 45 needed to be increased.
- At Station 45, provisions needed to be made for the positive transfer of stress from the exterior skin plates of the upper portion of the Arch above Station 45 to the concrete below this station. Proposed repair details were included as part of the study as a means of accomplishing this stress transfer.

- The exterior skin plates near Stations 44 and 45 needed to be changed from stainless steel to stainless clad steel, and additional transverse stiffeners needed to be added. The extent of this change above Station 44 was not fully determined at the time of the Bureau of Reclamation study.

The following calculations, analyses and test results were included as appendices to the December 1964 Bureau of Reclamation report, which were used to determine the structural deficiencies and recommend appropriate structural repairs.

- *Discussion and Determination of Wind Forces on the Jefferson National Expansion Memorial Arch*, by Robert H. Schaefer, Structural Engineer, June 8, 1964.
- Thermal Studies
 - Plate Temperatures
 - Thermal Stresses
- *Earthquake Loadings for the Jefferson National Expansion Memorial Gateway Arch*, by C.C. Crawford, March 20, 1964
- Laboratory Tests
 - Concrete Tests (Cylinder Tests of Concrete), July 31, 1964
 - Stainless Steel Tests (Stress-Strain Curves), October 9, 1964
 - Thermo-Gradient Test of Section, September 10, 1964
- Structural Model Tests
 - Model Study—Arch Wall Section Station 37, July 13, 1964
 - Experimental Stress Study—Diaphragm, May 28, 1964
 - Corner Bracing Study, by Ira E. Allen and Richard W. Ribbens, October 29, 1964
 - Corner Shear Study. This document is referenced in a previous report, but was not available for review during this study.
 - Review of Theories of Failure. This document is referenced in a previous

¹²² “Report on the Jefferson National Expansion Memorial Gateway Arch,” Department of the Interior, Bureau of Reclamation, Division of Design, Office of Chief Engineer (December 1964).

report, but was not available for review during this study.

- *Discontinuity at Station 37*, by Ira E. Allen and Richard W. Ribbens, October 1, 1964
- *Computer Analyses of the Jefferson Memorial National Expansion Memorial Arch*, by H. Walter Anderson through William H. Wolf, Power Plant Structures Section, December 10, 1964.
 - Structural Analysis of Arch in Completed Form
 - Structural Analysis of Arch during Construction

Aerodynamic Stability of the Jefferson National Expansion Memorial Gateway Arch, July 1965

A copy of this report is included in Appendix C. Wind tunnel tests, as requested by the Bureau of Reclamation, U.S. Department of the Interior, resulting from the review of the structural design calculations and drawings of the Gateway Arch were conducted on a full elastic 1:120 scale model of the Gateway Arch to determine the aerodynamic stability of the structure in the wind. As part of the initial analysis, the report on aerodynamic investigation on the original design, conducted by Dr. D. B. Steinman, was reviewed. The report stated the need for modifications and further aerodynamic tests to verify the effectiveness of these modifications. It was confirmed that these modifications were made to the design, yet no verification of their effect on the aerodynamic stability is documented in the archival materials reviewed. The wind tunnel tests were completed by the Bureau of Public Roads under a memorandum of understanding between the Bureau of Reclamation and the Bureau of Public Roads dated September 28, 1964, more than two years after the start of construction of the Arch.

Analytical determination of the waveforms and frequencies, coupled with the results from

the wind tunnel tests of the scaled model, were used to determine the response of the Gateway Arch to various wind conditions (including speed and direction.) While the Gateway Arch differed in important aspects from typical suspension bridges, the wind tunnel facility at the Bureau of Public Roads Fairbank Highway Research Station, designed for testing section models of suspension bridges, was selected to test the model of the Gateway Arch. Selecting this facility and testing procedures used for suspension bridges provided conservative results, which was deemed acceptable because of the risk to public safety.

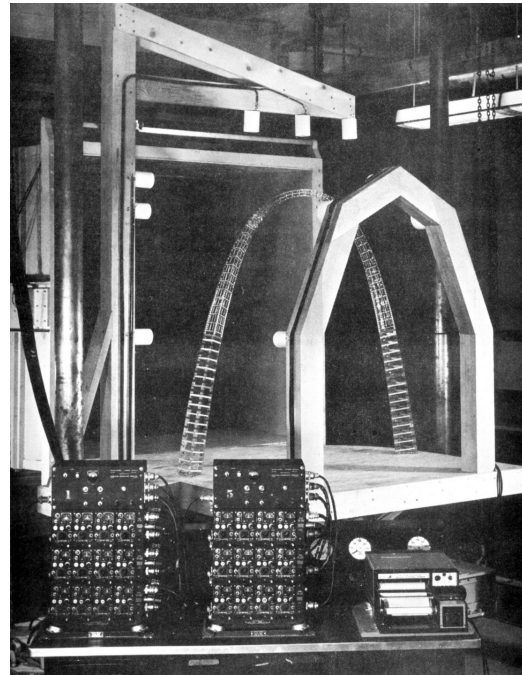


Figure 123. General view of wind tunnel instrumentation, with model of Arch oriented for 90-degree wind angle. Source: 1965 Aerodynamic Stability of JNEM Arch by Lloyd R. Cayes, Figure 7.

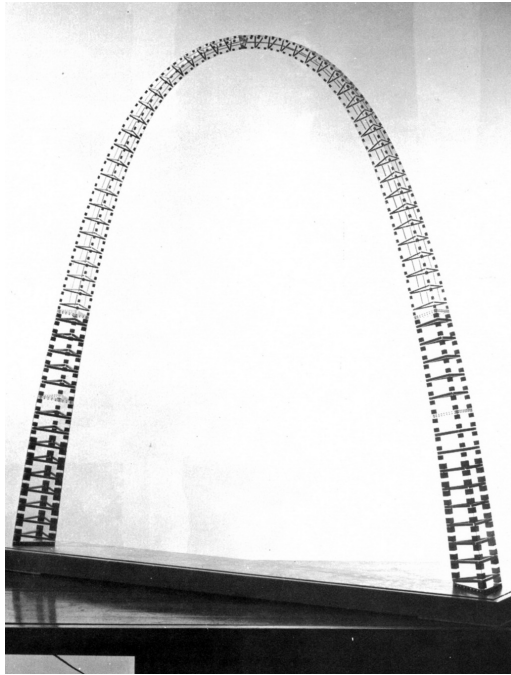


Figure 124. First model of Arch for wind tunnel testing. Source: 1965 *Aerodynamic Stability of JNEM Arch* by Lloyd R. Cayes, Figure 9.

The following conclusions were made as part of the model tests and evaluation of limited field tests during the aerodynamic study.¹²³

- Potentially dangerous oscillations of the model arch for both the completed Arch and during critical erection stages, just prior to closure, were highly probable under unfavorable wind direction and velocity.
- Critical wind direction is parallel to the Arch (north-south direction) and significant oscillations in the fundamental mode may be expected if wind velocities greater than 50 mph are sustained for significant periods.

¹²³ Memorandum from the Bureau of Reclamation, excerpts of conclusion from *Aerodynamic Stability of the Jefferson National Expansion Memorial Gateway Arch*, by Lloyd R. Cayes and Charles F. Scheffey, July 1965.

- The oscillations experienced during the wind tunnel study are such that they may be controlled by the introduction of additional damping.
- Levels of fluctuating strain did not produce the large damping absorption force required in the existing structural system.
- Conclusive evidence did not exist to consider the prototype arch model tests overly conservative in terms of public safety, thus it could not be concluded that the forces causing the oscillations in the wind tunnel studies would be greatly reduced at higher Reynolds numbers for the actual structure.

The following recommendations were made as part of the aerodynamic study in order to limit the potential excessive oscillations.

- The completed structure should be observed for any indication of aerodynamic oscillation.
- A system of artificial damping devices should be designed so that they may be fabricated and installed at the first signs of difficulty during construction, so as to limit excessive deflections.

Appendix A-Field Study of the Partially Completed Arch, April 1965

Strain and acceleration recordings were taken during April 1965 at two levels on the south leg of the Arch in connection with the movement induced by wind and construction operations. The construction of the south leg was complete to Station 30 (elevation 467 feet) at the time the study was conducted.

Bonded wire strain gauges were installed at the center of all three corner plates and parallel to the vertical corner lines on the interior face at Station 42. A 45-degree strain gauge rosette was installed on the extrados face midway between corners, also at Station 42. Pairs of accelerometers were

clamped to the structure near the strain gauges in the two extrados corners, in order to record separately the north-south and east-west accelerations. Two accelerometers were also installed at the top of the completed structure (Station 30) to measure north-south and east-west movement. During the visual observations of the interior legs of the Gateway Arch for the current study, no remnants of these strain gauges were observed. It is unlikely these strain gauges would be visible, as they were installed during construction and most likely removed prior to coating the interior steel skin.

The following results were obtained from the field study with regard to actual stresses and strains experienced by the partially constructed Arch. The live loads applied to the structure during the field investigation were quite low because of low wind velocities and caution taken when using the hoist. The responses were therefore quite small, thus, no comparisons could be made of other values between the prototype and actual field measurements, aside from fundamental frequency, which was measured as 0.89 in the field and 0.91 for the aerodynamic model testing. The damping and stress data measured also provide some insight into the probable behavior of the completed structure.

Other Documentation

- Memo from Werner Gottschalk of Severud-Elstad-Kruger-Associates, dated June 29, 1962. This memo references the proposed variation of the stiffener plate layout, because of the interference of the 3/8 inch stiffener bolts with the corner of the inner skin plates. The proposed variation was found acceptable, yet indicated the following requirements because of the change to the stiffener plate layout.
 - The post-tensioning tendons would be affected, thus requiring recalculation of the camber due to post-tensioning.
 - The stiffeners would die out at the edges and the center line of the arch
 - The arrangement of the steel sections above Station 45 needed to be clarified because of the proposed changes.
 - One of the concerns indicated that verifying the location of the post-tensioning rods and the stiffener plates would be required prior to proceeding with the placement of the foundation above elevation -34.0 to ensure that these locations match, so that the camber can be analyzed.
- Follow up memo from Bruce Detmers of Eero Saarinen and Associates, indicating the angled installation of the bolts as proposed by MacDonald Construction is preferred in the lower portion of the Arch, but suggested that MacDonald lay out the entire stiffener pattern prior to making a decision regarding the corner detail.
- Memo from Fred Severud of Severud, Perrone, Fischer, Sturm, Conlin and Bandel Consulting Engineers, to Eero Saarinen and Associates dated April 28, 1965 regarding the "Interim Report on Investigation of the Aerodynamic Stability of the Jefferson National Expansion Memorial" prepared by the Bureau of Public Roads. This memo reviews the aerodynamic study point by point and breaks down the logic used, assumptions made, and the inconsistencies between the model measurements and the actual Arch construction. Mr. Severud points out that the wind conditions utilized in the tunnel could never exist, because of the effect of natural turbulence (wind roughness) will be greatly affected by the ground conditions. Mr. Severud makes the statement that the only way to determine accurate behavior of the Arch, based on the uncertainties he elaborates on from the

Bureau's report is to take measurements from the Arch, once the erection truss is in place. His ultimate recommendation is to continue with construction of the Arch, exactly as designed, as he finds no error in the structural design, and field measurements should be taken when the erection truss is in place. He also indicated that any additional dampening devices added would be purely for human comfort and not required by the design.

- Memo from the Bureau of Reclamation prepared by Messers, Rose and Orlander dated April 13, 1965 regarding the "Interim Report on Investigation of the Aerodynamic Stability of the Jefferson National Expansion Memorial" prepared by the Bureau of Public Roads. The memo confirms the Bureau's concern regarding the aerodynamic stability of the Arch and summarizes the effect of the Bureau of Public Roads' report on the findings contained in the Bureau of Reclamation's December 1964 report on the structural adequacy of the arch. The memo reaffirms the previous findings by the bureau that the original design above station 45 is inadequate and the sections constructed above this portion should be dismantled. Ultimately, the recommendation is that continuation of the construction on the Arch as originally designed is unsafe.
- Follow up memo from Fred Severud of Severud, Perrone, Fischer, Sturm, Conlin and Bandel Consulting Engineers, to Eero Saarinen and Associates dated May 11, 1965 regarding the memo from the Bureau of Reclamation prepared by Messers, Rose and Orlander dated April 13, 1965 regarding the "Interim Report on Investigation of the Aerodynamic Stability of the Jefferson National Expansion Memorial" prepared by the Bureau of Public Roads. This memo references much of the information from the April 28, 1965

memo and comes to ultimately the same conclusions.

Condition Assessment: Foundation

Portions of the interior concrete foundation walls accessible from the tram load zones and maintenance spaces for both the north and south legs were inspected. The observed portions of the concrete foundations are in serviceable condition. The following conditions were observed during the visual review of the concrete foundations supporting the base of each leg of the Arch, as shown in Figures 125 through 132.

Minor cracking exists at isolated locations throughout the concrete foundation walls of both the north and south legs (Figures 127 and 128).

At some of the hairline cracks white efflorescence staining was observed leaching from the cracks, which usually is an indication of moisture migrating through the concrete (Figure 129).

Minor evidence of moisture staining was observed on the concrete foundation walls of both the north and south legs (Figure 130).

Isolated locations of the concrete ceiling, visible in the tram load zone only, exhibit spalling and exposed staining caused by corrosion of the reinforcing bars (Figure 131).

Surface corrosion was present on the steel plates at the transition from the steel triangular cross section to the concrete foundation walls (Figure 132).



Figure 125. View of concrete walls at tram loading zone area. Source: WJE, 2008.



Figure 126. View of concrete walls at tram load zone. Source: WJE, 2008.



Figure 127. View of steel framing supporting the tram. Source: WJE, 2008.



Figure 128. View of minor shrinkage cracks at the concrete footings. Source: WJE, 2008.



Figure 131. View of corrosion stains and white staining indicative of moisture infiltration. Source: WJE, 2008.



Figure 129. View of shrinkage cracks and moisture infiltration at concrete footings. Source: WJE, 2008.



Figure 132. View of minor surface corrosion at the transition plate between the interior steel skin and the concrete foundation walls. Source: WJE, 2008.



Figure 130. View of shrinkage cracks present on the concrete ceiling of the tram load zone. Source: WJE, 2008.

Condition Assessment: Exterior

Refer to the Exterior description section, above, for a review of the observed exterior conditions. It is difficult to assess when the existing staining on the exterior Arch surfaces first became apparent due to the lack of maintenance records and periodic milestone photographs. Comparing the current observations with early photos collected in *The Gateway Arch: An Architectural Dream* and the JNEM archives, the exterior skin has definitely altered since initial completion of the Arch. However, it is not possible to accurately determine the rate of visual alteration. Additional comparison of archival photographs and baseline photographs taken during this study with future observations may be useful in determining the progression of the staining.

Historic Use of Stainless Steel

There are numerous examples of buildings and monuments of large scale that have incorporated exterior stainless steel cladding. The following examples may provide sources for further research and understanding of the fabrication, maintenance, and long term weathering of stainless steel to guide future treatment of the Arch exterior skin. The Chrysler Building, completed in New York in 1929, utilized Nirosa metal for its spire, a stainless steel alloy developed by Krupp, the German steelworks. Nirosa metal had approximately 18 percent chromium and 8 percent nickel, making an alloy not unlike grade 304 as defined in current ASTM standards. It is significant to note that the spire of the Chrysler was cleaned in 1995. The metal portions of the facades of both the Lever House in New York (1952) and the Inland Steel Building in Chicago (1957), both designed by Skidmore, Owings and Merrill, used stainless steel plate, but the alloy has not been determined. The Motherland Statue in Kiev, U.S.S.R. (present-day Ukraine), was

completed in 1981. The figure is made of chrome-nickel steel and stands 200 feet above its base.

Corrosion of Stainless Steel

Atmospheric corrosion is an electrochemical process that requires three key components: an anode; a cathode; and an electrolyte. A corroding metal site that consequently generates electrons acts as an anode, while the surrounding metal under a wet film acts as a cathode, which reacts by consuming the electrons generated by the anode. The electrolyte, formed from condensation and/or deliquescence, provides the ionic transportation to complete the circuit such that a current flows from the anode to the cathode. Without the presence of water or another electrolyte, corrosion will not occur. Therefore, there is a risk of corrosion occurring as soon as a water film or droplets form on a metal surface.

Under normal conditions, stainless steel exhibits a passivity approaching that of noble metals (i.e., metals that are highly resistant to oxidation and corrosion). However, when the oxide film is breached and prevented from redeveloping by aggressive elements such as chloride ions, the stainless steel surface becomes active and can corrode like iron. The following types of corrosion are known to occur with stainless steel.

Pitting Corrosion: Widespread, superficial, localized corrosion observed on surfaces near grade may be attributed to the use of de-icing salts, vandalism, possibly aggressive cleaning methods used near the base of the Arch, and weather. Airborne particles with aggressive ingredients may have collected on the surface of the Arch since the final cleaning during construction in 1965–1966. This accumulation is consistent with the staining observed, since the top of the Arch is essentially free of corrosion and is regularly washed by

rainwater, removing the aggressive particles. In addition, the south and north faces of the Arch exhibit little corrosion, possibly resulting from their slightly upward orientation, which is subjected to maximum rain exposure and run off.

Intergranular Corrosion: Corrosion of welding lines was observed near the ground of the Arch. Such corrosion is probable higher on the structure as well. In addition, streaking from the field welds, as compared to the shop welds, was commonly noted.

Galvanic Corrosion: This type of corrosion involves the interaction of dissimilar metals. Galvanic corrosion is not often seen on architectural metals, and austenitic (corrosion-resistant) stainless steel will generally resist this type of corrosion. For the exterior skin, only the stainless steel is exposed and thus this type of corrosion should not occur. However, there are tens of thousands of locations where stainless and carbon steel are directly in contact between the skins. There are two different scenarios in which this condition occurs: at the lower part with the concrete fill and upper section containing the empty space.

Causes of Staining and Discoloration

The causes of staining on the exterior faces of the Arch cannot be determined definitively unless these surfaces can be observed close up on the exterior and a more comprehensive investigation is carried out. However, there are numerous hypotheses that can be formed from the initial inspection as discussed below.

The stainless steel skin is discolored and stained to different degrees depending on surface conditions, and represents a variety of phenomena. The rain-washed south to north surface has minimum staining, while the other faces have minor to moderate corrosion. The extent of possible corrosion is difficult to determine at higher elevations due to access

limitations. It is possible that corrosion at welds or at contaminated areas is taking place aggressively. Close-up inspection is required to confirm these conditions.

As the structure is subject to dynamic stress cycles, there is a possibility that welds have failed locally, generating points of water leakage into the interstitial space. Corrosion products of carbon steel may then have stained the stainless steel surface.

For the lower concrete-filled cross-section, especially the section near the 300 foot level, it is likely that the concrete is wet. In this situation, carbon steel will be passive as long as the concrete retains its characteristic high pH and has little chloride. Consequently, the carbon steel and the passive stainless steel will have similar electrochemical potential, and thus galvanic corrosion is not likely a problem. If the concrete was cast with chloride accelerators, however, galvanic corrosion is a potential problem. No records indicating the use of chloride accelerators in the concrete fill have been found during archival research for this project.

For the upper cross-sections having empty space in between the skins, the carbon steel will likely corrode due to condensation, while the stainless steel will remain passive. In this case, the two types of steel will have a potential electrochemical difference on the order of a few hundred millivolts, creating galvanic corrosion cells. Consequently, corrosion of internal carbon steel is expected to be accelerated. Such galvanic corrosion could be less destructive than one might expect. It is speculated that it will take a long time for such corrosion to induce any integrity concern.

Upon completion of original construction, the majority of the water used in the concrete mix was not able to evaporate, as space between

interior and exterior skins was sealed by welding all of the segments together; thus, it is assumed that the voids between the interior plate and the concrete fill were at some point filled with water. As no inspection openings were created as part of the initial investigation, the presence of water in this void could not be confirmed during this investigation. An inspection between the skins is necessary to document actual conditions, assess the extent of corrosion, and evaluate related concerns.

Corrosion of grade 304 stainless steel is mainly due to accumulation of dust, dirt, and salt. Therefore, the key to long term preservation of the stainless steel exterior panels is to keep the surfaces clean. The fact that the most exposed and therefore rain-washed top surfaces (south face of south leg, north face of north leg, and top) exhibit little corrosion or staining testify to this statement. Stainless steel with a clean surface is typically passive and corrosion-free in ambient conditions. Initiation and propagation of such corrosion is related to both weather and air quality.

Condition Assessment: Interior

A close-up visual inspection, inclusive of sounding for delamination, was performed on the interior surface of the segment plates of the north and south legs of the Arch from the interior stairways. Maintenance personnel from JNEM provided BVH/WJE with access for these inspections, which were conducted from the access stairways within each leg of the Arch.

During the investigation of the interior of both legs of the Arch, maintenance personnel offered comments about their long-term observations of conditions inside the Arch and information about recently completed repair and maintenance work. The most significant maintenance concern and repeated comment from the different maintenance staff personnel was associated with the micro-climate inside the legs of the Arch. Refer to the discussion of this issue below.

Structural Assemblies

Sounding was performed on the surface of the interior plate at the locations where corroding fastener heads were found. At these locations, delamination (an air gap) exists between the concrete and the interior skin. The hollow sounding areas were primarily concentrated at the corroding fasteners and along the length of the stiffener angles, where it was presumably difficult to consolidate the concrete around these elements (Figures 133 and 134). Refer to drawing S107-B in Appendix I.

At isolated fastener heads remaining from the erection framing below Segment 45, which marks the top of the concrete pour, cement paste has leached out at some of the bolt heads (Figures 135 and 136).



Figure 133. The interior carbon steel skin was sounded to identify delaminations. Source: WJE, 2008.



Figure 136. View of cement paste leaching out at bolt heads. Source: WJE, 2008.



Figure 134. Delamination identified in interior carbon steel skin by sounding, highlighted by dotted lines. Source: WJE, 2008.



Figure 137. View of bent and deformed shanks of the tie rod bolts. Source: WJE, 2008.



Figure 135. View of cement paste leaching out at bolt heads. Source: WJE, 2008.

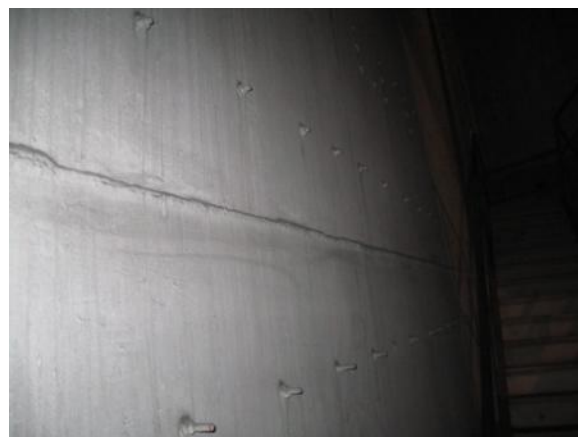


Figure 138. View of variable extensions at the 3/8 diameter stud bolts. Source: WJE, 2008.



Figure 139. View of bolt installation. Source: JNEM archives, image V106-3973.

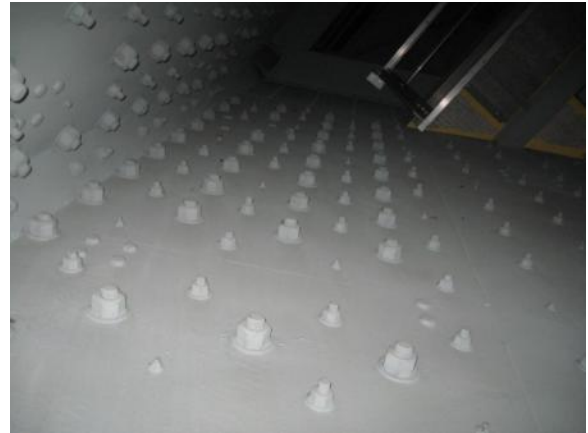


Figure 142. View of bolt layout out at concrete fill transition. Source: WJE, 2008.



Figure 140. View of the bolt installation. Source: JNEM archives, image V106-3977.



Figure 143. View of bolt layout out at concrete fill transition. Source: WJE, 2008.



Figure 141. Typical bolt layout of below station 45. Source: WJE, 2008.



Figure 144. View of plug weld layout between Stations 37 and 45. Source: WJE, 2008.

At isolated locations, some of the shanks of the stud bolts connecting the diagonal tie rods to the vertical stiffener angles are bent and deformed, as shown in Figure 137. Variable extensions exist at the 3/8 diameter stud bolts, fastening the diagonal tie rods and the vertical stiffener angles together, as shown in Figure 138. The bolt sizing and placement below Station 46 at the concrete transition varies (Figures 139 and 140). The bolts are 3/8 inch diameter stud mild steel and high-tension steel, with 2 by 3/8 inch straps below Station 45. The typical bolt layout away from the corners is shown in Figure 141. Also refer to drawings S107 and S107-B in Appendix I. The transition layout for the bolts between Stations 45 and 46 includes 7/8 inch diameter high-tension stud bolts and 1-1/2 inch diameter bolts at the corners, with tighter spacing (Figures 142 and 143). Also refer to drawing S108 in Appendix I.

Above the concrete termination, Station 45, the bolt spacing and patterns as well as plug weld layout and spacing varies. Between Stations 37 and 45, the interior 1/4 thick stiffener plates, diaphragms, and 1/2 inch diameter diagonal bars are plug welded to the interior skin plate with 3/4 inch plug welds spaced approximately 6 inches on center. Refer to drawing S113 in Appendix I. The thicker interior skin corner plates (1-3/4 inch thick) are bolted to the interior stiffener plates with 1/2 diameter high strength bolts (Figure 144 and 145). Also refer to drawing S113 in Appendix I; two versions of this drawing exist, 113 and 113B: one shows only bolts and one shows plug welds. The exterior skin plates are fastened to the internal stiffener plates and diaphragms with double spot fillet welds spaced at a maximum of 5 inches on center. At each station line, transverse plates approximately 3/8 inches thick are welded to the inside face of the exterior plate and a closure plate at the interior skin.



Figure 145. View of plug weld layout between Stations 37 and 45. Source: WJE, 2008.



Figure 146. View of variable field welds. Source: WJE, 2008.



Figure 147. View of plug welds from Station 0 to 28. Source: WJE, 2008.

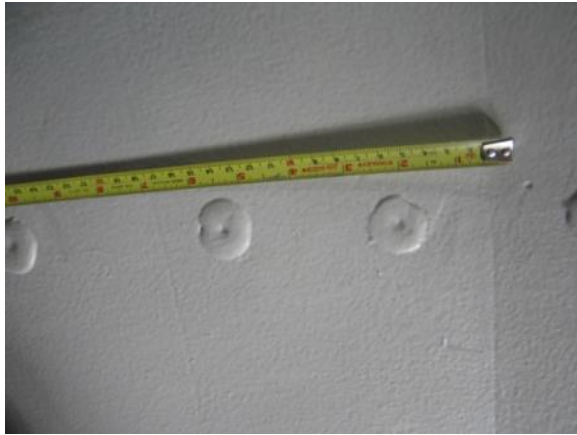


Figure 148. View of plug welds between Stations 0 and 28. Source: WJE, 2008.



Figure 149. View of vertical shop welds at Station 28. Source: WJE, 2008.

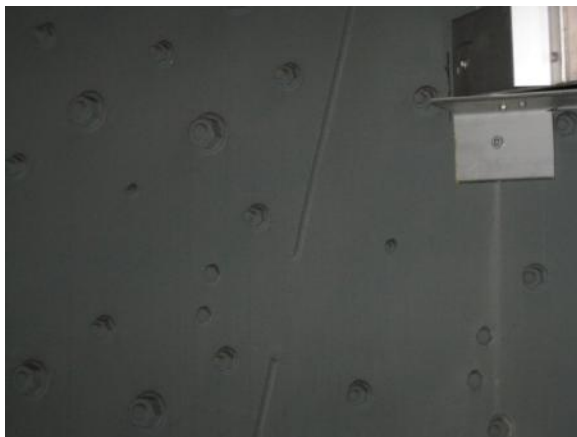


Figure 150. View of vertical shop weld, with portions ground flush. Source: WJE, 2008.

From Station 28 to Station 37, the interior 1/4 thick stiffener plates, diaphragms, and 1/2 inch diameter diagonal bars are plug welded to the interior skin plate with 3/4 inch diameter plug welds spaced at approximately 6 inches on center. Refer to drawing S114 in Appendix I. The thicker interior skin corner plates (1-3/4 inch thick) are bolted to the interior stiffener plates with 1/2 diameter high strength bolts (Figure 146). Also refer to drawing S113 in Appendix I. The exterior skin plates are fastened to the internal stiffener plates and diaphragms with double spot fillet welds, spaced at a maximum of 5 inches on center. At each station line, transverse plates approximately 3/8 inches thick are welded to the inside face of the exterior plate and a closure plate at the interior skin.

From Station 0 to Station 28, the interior 1/4 thick stiffener plates, diaphragms, and 1/2 inch diameter diagonal bars are plug welded to the interior skin plate with 3/4 inch plug welds spaced at approximately 6 inches on center (Figures 147 and 148). The thicker interior skin corner plates (1-3/4 inch thick) are bolted to the interior stiffener plates with 1/2 diameter high strength bolts. The exterior skin plates are fastened to the internal stiffener plates and diaphragms with double spot fillet welds, spaced at a maximum of 5 inches on center. Transverse plates approximately 3/8 inches thick are welded to the inside face of the exterior plate and a closure plate at the interior skin, at each station line.

The shop welds have been ground flush in some locations and appear to be single pass full penetration groove welds. The shop welds are typically the vertical welds between the plates that make up one Arch segment (Figures 149 and 150).

The field welds, visible on the interior, exist between segments and are multi-pass full penetration groove welds. The weld reinforcement above the plane of the plate is not flush with the plate surface. The multi-pass welds vary in size, which may have been necessary to accommodate for tolerance variances (Figures 151 through 153).

Portions of the steel framing used during erection remain in place, as evidenced by the I-beams whose flanges are bolted to the interior plates at approximately quarter spans of the height of each segment. The steel left in place was notched/torched in many locations to accommodate the tram system and stair components, thus is thought to have been used only for stiffening purposes during erection and construction. Figures 154 through 156 show typical locations of the erection framing on the interior of the Arch.

These structural framing members are visible in many of the construction photos (Figure 157). Ken Kolkmeier confirmed that the steel wide flange members remaining at portions of the interior of Arch segments were used to brace triangular segments during erection until they were permanently set. Mr. Kolkmeier noted that the steel was left inside for the most part but was originally intended to be removed and salvaged. He also noted that as erection progressed it became more difficult and costly to remove these steel members and that they were consequently left in place, especially at the top of the Arch. The steel members were also useful for workers to use as a platform from which to work, as shown in Figure 158.¹²⁴



Figure 151. View of variable field welds between Stations 39 and 40. Source: WJE, 2008.



Figure 152. View of variable field welds between Stations 36 and 37. Source: WJE, 2008.



Figure 153. View of variable field welds between segments at extrados corner. Source: WJE, 2008.

¹²⁴ Ibid.



Figure 154. View of remaining erection wide flange framing at interior skin plates. Source: WJE, 2008.



Figure 155. View of remaining erection wide flange framing at interior skin plates. Source: WJE, 2008.



Figure 156. View of remaining erection framing at interior skin plates. Source: WJE, 2008.



Figure 157. Underside of an Arch section during assembly, Note the interior stiffening elements. Source: JNEM archives, image V106-3977.



Figure 158. Interior of Arch being painted, with workers using the remaining wide flange sections as platforms from which to perform the work. Source: JNEM archives, image V106-4167.

Staining Related to Water Leakage and Condensation

Moisture staining, as evidenced by numerous streaks and some white residue, was observed. The white residue is present at random bolt heads/nuts. This condition was more prevalent at the small 3/8 diameter stud bolts below Station 45, which tie the interior and exterior plates together (Figures 159 and 160).

Streaks were visible on many of the painted steel plates inside both legs of the Arch, suggesting significant condensation formation that could form water droplets and create streaking marks. See Figures 161 through 163 for typical water marks and streaking observed on the interior carbon steel plates.

In the south leg, streaking and evidence of moisture condensation is more consistently present (Figures 164 through 166).

Streaking, evidence of previous standing water, and surface corrosion were apparent on the horizontal stair surfaces of the stair landings and treads, and any horizontal surfaces where moisture can accumulate. Evidence of moisture on the stair treads/landings appears to be more significant at the south leg. Figure 167 shows an example typical corrosion at painted steel stair treads and landings.

JNEM maintenance personnel also reported that the louver ventilators at the top of the Arch at Segment 5 are sources of active water leaks.



Figure 159. View of moisture staining at bolt heads. Source: WJE, 2008.



Figure 160. Typical view of moisture staining at bolted connections. Source: WJE, 2008.



Figure 161. View inside north leg of deposits/staining from water on the interior surface of the structure. Source: WJE, 2008.



Figure 162. View of streaks visible on many of the interior painted steel plates. Source: WJE, 2008.



Figure 165. View of streaks visible on many of the interior painted steel plates in the south leg. Source: WJE, 2008.



Figure 163. View of minor streaks visible on many of the interior painted steel plates. Source: WJE, 2008.

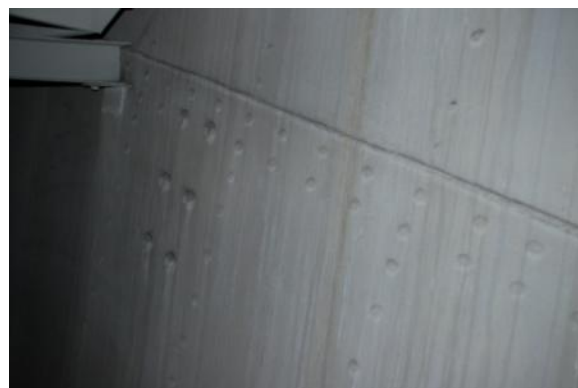


Figure 166. View of streaks visible on many of the interior painted steel plates in the south leg. Source: WJE, 2008.



Figure 164. View of streaks visible on many of the interior painted steel plates in the south leg. Source: WJE, 2008.

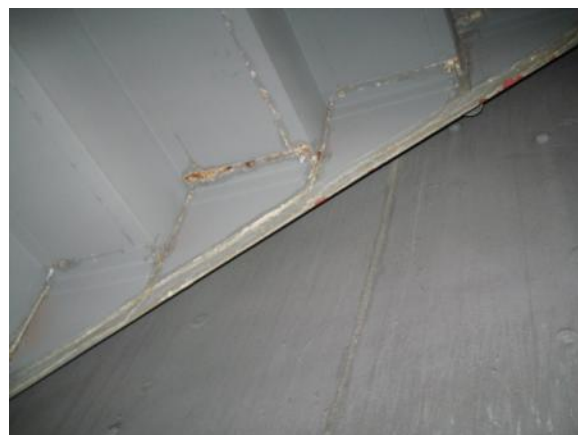


Figure 167. View of evidence of standing water, as indicated by surface corrosion at underside of stairs. Source: WJE, 2008.

Micro-Climate Conditions within the Arch

The interior of the Arch, excluding the fully conditioned observation deck and museum, has a unique micro-climate that is caused by the makeup of the monument. This interior micro-climate was apparent even before the Arch was fully completed. Changes in temperature and solar conditions cause the inner walls to heat and cool rapidly, creating condensation in liquid or frost form, as reported by JNEM staff. At times, it is possible for dripping condensate (“rain”) or visible water vapor (“fog”) to occur inside the Arch legs.

The interior micro-climate of the Arch is a function of the ambient air temperature, the surface temperature of the metal structure, and the dew point. Ambient air temperature is the actual air temperature, either interior or exterior, as measured by a dry bulb thermometer. Surface temperature is the actual temperature at the exposed interior or exterior surface of the structure. Dew point is the temperature at which the moisture contained within the air as vapor can condense to form liquid water. The higher the dew point, the higher the moisture content of the air (that is, the higher the humidity). If the dew point of the air on the interior of the Arch is relatively high and the air or surface temperature quickly drops, water droplets can form on the cooling surface of the Arch. This occurs, for example, when night or a cold rain follows a hot and humid day. This condensation process can result in the development of fog, frost, and even precipitation within the Arch. Condensation can also form as water droplets or continuous films on metal surfaces. Wet metal surfaces promote corrosion, as further discussed below.

The phenomenon of deliquescence can exacerbate issues of condensation-related deterioration of metal surfaces. Many salts can significantly increase the possibility for metal

surfaces to become wet, even at low dew points, because the salts tend to draw moisture from the air. For example, on a table with particles of table salt (NaCl) at an average dew point, while most of the surface is dry, the surface beneath the NaCl is wet.

Unfortunately, deliquescence often results in concentrated solutions, which are very aggressive. If salts are present on the Arch surfaces, those surfaces will remain wetter than those surfaces where salts are not present.

At the south Arch leg, above and below the concrete fill line (Station 45) the carbon steel plates on the three interior surfaces varied in temperature, as determined from simple tactile analysis. By touching the various plates, it was noted that in the afternoon the plates on the west side above the concrete fill line were noticeably warmer than those on the east, indicative of radiant heating from the sun on the exterior skin plates. Furthermore, the southward facing surface was noticeably warmer than the eastward or westward facing surfaces. In segments below the concrete fill line, the wall surface temperature variation was not as significant. This observation indicates that the concrete fill acts as insulation and a barrier between the interior and exterior skin. This thermal mass allows for a slower and more uniform distribution of heat from the exterior stainless steel skin surface to the interior carbon steel plates. This difference in interior surface temperature variations results in a different microclimate in the lower portion of the Arch compared to the upper portion.

Some of the interior condensation may be attributed to malfunctioning steam piping, which allows additional moisture into the interior portions of the legs, raising the relative humidity of the interior. Refer to the Mechanical, Electrical, and Plumbing assessment, Arch Leg Air Handlers section.

Temperature and Humidity Monitoring

As part of the initial investigation for this study, temperature and relative humidity readings were recorded using a digital handheld psychrometer in the north leg of the Arch on November 12, 2008 (Figure 168). The readings are included in Table 1 in Appendix D. There is some variance of the temperature and relative humidity readings between the conditioned observation area at the top of the Arch and the museum areas at the base as compared with the interior air temperature of the north leg near the concrete termination at Station 45. These measurements show that the dew point (a measure of absolute water within the air) was highest in the museum and then dropped markedly, but remained constant within the leg. The dew point outside was lower than that inside the leg, indicating that the interior air was more humid.



Figure 168. View of digital psychrometer used for measuring temperature and relative humidity in the Arch. Source: WJE, 2008.

To understand the conditions within the interior of the Arch, a longer term temperature and relative humidity monitoring program was conducted as part of this investigation. For purposes of this investigation, small temperature and relative humidity monitoring devices were installed in three locations inside the south leg and in one exterior location. The

temperature and relative humidity monitors were removed on September 1, 2009, and the data were compiled. A more detailed description of the monitoring system, including graphs of the data collected, is included in Appendix D.

Four dry bulb air temperature (DBT) and relative humidity (RH) monitors were installed within the Arch in the following locations:

- Monitor 1: exterior
- Monitor 2: tram load zone within the concrete footings in the south leg
- Monitor 3: Station 14 of the south leg of the Arch
- Monitor 4: Station 47 (above the concrete filled portion) of the south leg of the Arch

The monitors were in place and recording continuously at ninety minute intervals for the period between November 20, 2008, and July 26, 2009.

Since the monitors were measuring air temperature rather than the surface temperature of the interior plates and/or concrete footings, the potential for condensation would still exist even if the relative humidity was less than 100 percent or the dew point temperature was below the dry bulb temperature. If the temperature and dew point are within 5 degrees of one another, there is potential for condensation to occur.

Monitor 1: One hundred seventy four occurrences were recorded when the RH reached 100 percent. These readings typically occurred during the months of May, June, and July, with some isolated instances in December and April. The interior DBT was compared with the exterior DBT, to determine if condensation could potentially occur.

Monitor 2: At the seven occurrences when the RH was greater than 80 percent on June 16–

19, 2009, the difference between the exterior and interior air temperatures at the concrete footings varied on average approximately 4 degrees, with the exterior temperatures being higher. Only one occurrence of a difference of less than 5 degrees between the interior air temperature and interior dew point occurred on June 19 at 1 a.m., which would indicate a potential for condensation. Only one occurrence of a difference of less than 5 degrees between the exterior air temperature and interior dew point occurred on June 17. No occurrences were recorded where the interior dew point was less than the exterior temperature.

Monitor 3: At twenty-six occurrences when the RH was greater than 80 percent during late April, May, June, and July, the difference between the exterior and interior DBT varied on average approximately 4 degrees, with the exterior temperatures being lower. There were five occurrences of a difference of less than 5 degrees between the interior DBT and dew point (three on May 26 and two on July 5). Twenty-six occurrences of a difference of less than 5 degrees between the exterior air temperature and interior dew point were recorded. There were three occurrences where the interior dew point was less than the exterior temperature, on April 30, May 16, and May 26. All of these instances indicate a potential for condensation.

Monitor 4: At the three occurrences when the RH was greater than 80 percent on July 5, the difference between the exterior and interior DBT at the concrete footings varied on average approximately 6.5 degrees, with the interior temperature being lower. A difference of less than 5 degrees between the interior DBT and interior dew point did not occur. Three occurrences of a difference of less than 5 degrees between the interior DBT and interior dew point occurred on July 5. No occurrences were recorded when the interior

dew point was less than the exterior temperature.

Based on the readings from the interior monitors 2, 3, and 4 as compared with the readings from monitor 1 located on the exterior, it is possible that condensation could occur within the interior legs of the Arch. The monitor in the upper portion of the Arch (above the level of the concrete fill) indicated that interior surfaces in this location had the greatest potential for condensation, as indicated by the number of occurrences within the 5 degree temperature differential between air temperature and dew point. Most of these occurrences were recorded in late May, June, or early July, in the early hours of the morning. Typically, the readings within a 5 degree temperature differential, or potential for condensation, occurred between 11:30 p.m. and 7:00 a.m.

Corrosion

At the bottom of the north base Segment 71, the top of the concrete foundation, corrosion was found at intrados corners where water may have collected for prolonged periods. See Figures 169 and 170 for examples of corrosion occurring at the base segment in the north leg due to pooling water. The corrosion of the south leg at the base segment is not as severe as that of the north leg. A view of the corrosion of the south leg shown in Figure 171.

Isolated surface corrosion exists at a number of locations on the interior skin plates, both above and below the termination level of the concrete fill. The corrosion has typically initiated at bolt heads/fasteners in the carbon steel plate or at joints within the stairs where water can collect and the paint coating is more likely to have defects (Figure 172). Typically, many of the shanks of the small 3/8 inch diameter stud bolts were observed to be more severely corroded than the heads of the high strength bolts (Figures 173 through 175).

At Segment 45, which marks the top of the concrete pour, some of the bolts, washers, and nuts are corroding, indicating that there may be moisture trapped in the interstitial space between the two skin plates. It was also observed that many of the corroded bolts had been circled and numbered, indicating that they were the focus of previous surveys, though according to JNEM personnel there appears to be no record of any such survey.

At locations where surface corrosion exists on the interior plates, only minimal section loss was observed on the interior skin. The actual amount of section loss could not be confirmed during this study because both sides of the interior steel plates are not visible (Figure 176).



Figure 169. View of corrosion occurring at the base segment in the north leg. Source: WJE, 2008.



Figure 170. View of corrosion occurring at the base segment in the north leg due to pooling water. Source: WJE, 2008.



Figure 171. View of corrosion of the south leg at the base segment is not as severe as at the north leg. Source: WJE, 2008.



Figure 172. Surface corrosion on the horizontal surface of the temporary erection framing and stair landings. Source: WJE, 2008.



Figure 173. Many of the shanks of the small diameter stud bolts were corroding more severely than the heads of the high strength bolts. Source: WJE, 2008.



Figure 174. View of corroding bolts at the concrete termination, Station 45. Source: WJE, 2008.



Figure 175. View inside north leg showing corrosion at bolt heads/fasteners in wall. Source: WJE, 2008.



Figure 176. Example of limited surface corrosion observed on interior carbon steel plates. Source: WJE, 2008.

Surface corrosion was apparent on stair landings and treads where water would naturally collect (Figure 177 and 178).

For the coated carbon steel surfaces of the Arch interior, the presence of water on unprotected steel is sufficient to initiate corrosion. Water resulting from condensation is evident throughout the interior of the Arch, as a result of the micro-climatic conditions that are unique to this structure. Corrosion of the carbon steel Arch interior is localized at areas exhibiting coating conditions such as holidays (small voids) or delaminations (detachment or peeling). Localized corrosion may also occur where condensate water has collected aggressive particles or dust from the air (leading to deliquescence of salts).

It is likely that corrosion is taking place in the interstitial space between the inner carbon steel and outer stainless steel walls. Evidence of this corrosion was observed at steel anchors and stud bolts that pierce the inner carbon skin. Condensation may also be occurring in the interstitial space. The micro-climate between the two skins is not known, and it may be as or more conducive to condensation formation as the interior of the Arch.

Even if both skins remain well sealed and relatively impervious to ambient air infiltration, large amounts of water vapor may be present due to the curing of the concrete fill within the lower Arch segments up to the 300 foot level. Water can also probably enter the interstitial space through the louvers at Segment 5. While the welding was done well, the Arch is subject to dynamic stress cycles due to temperature changes and wind, and these cycles could have induced local failure of welding and thus form points of water entry. Also, during construction water may have entered the interstitial space when it rained, and the water would not have drained completely from the space.



Figure 177. View inside south leg of corrosion along underside of stair treads. Source: WJE, 2008.



Figure 178. View inside north leg of corrosion at stair landings. Source: WJE, 2008.

Even below the 300 foot level, it is likely that small voids exist between the steel and stainless steel interior and exterior skins and the concrete fill, due to the shrinkage of concrete as it cures and the difficulty of obtaining a full adhesive bond between the smooth metal surfaces and the concrete fill. These voids may at one point have been filled with water. Upon completion of construction, there would have been no way for the majority of the water in the mass concrete to easily evaporate. As no inspection openings were made during the investigation, the current presence of water in this void could not be confirmed.

For the carbon steel, the greatest threat is water from leakage, condensation, and deliquescence. Coring into the interstitial space and close inspection is needed to clarify such concerns.

Coating

Inside the Arch, the carbon steel is typically covered by a grey paint coating. The coating is typically in good condition; however, some coating distress has occurred, including significant delamination and repainting in areas of previous paint loss showing area that was likely repainted due to distress in the paint coating on the steel (Figure 179).

Inside the Arch, the carbon steel plates are generally coated with a grey paint and primer. Refer to Appendix B for a more detailed description of the coatings. Graphite shavings cover much of the horizontal surfaces on the interior of the Arch legs (Figure 180); these deposits are a byproduct of the sliding carbon shoes used to maintain electrical and signal continuity with the moving tram and/or the motor brushes of the direct current generators. JNEM maintenance staff indicated that a large central vacuum system is used to clean the interior legs of the Arch on a biannual basis (Figures 181 and 182).



Figure 179. View inside south leg of repainted area on carbon steel. Failure of the original paint layer in this area may have been due to rusting of the steel. Source: WJE, 2008.



Figure 180. View of graphite shavings remaining on the bolt heads. Source: WJE, 2008.



Figure 181. View of tram gears and cables. Source: WJE, 2008.



Figure 182. View of tram gears and cables. Source: WJE, 2008.

Maintenance and Alterations

To address condensation within the Arch, JNEM maintenance personnel have previously fabricated and installed an ad-hoc water collection system in the upper portion of each leg. The system is composed of wicks running down the lower inside vertex of the Arch cross-section. Water is collected in barrels located at a stair landing (Figures 183 and 184).



Figure 135. View inside south leg of wick used in water collection system. Note new orange paint coating at locations where corrosion caused distress in original coating. Source: WJE, 2008.



Figure 184. View inside south leg of water collection barrel used in conjunction with the wick system. Source: WJE, 2008.

Mechanical, Electrical, and Plumbing

Heating, Ventilating, and Air Conditioning

Arch Leg Air Handlers

The north and south legs of the Arch are currently served by large blow-through type built-up air handlers, which are located in the north and south mechanical rooms adjacent to the visitor center/museum. These built-up air handlers are original to the construction of the Arch and its associated mechanical systems. Archival construction documents indicate that the air handlers are manufactured by Trane. The air handlers utilize a hot deck/cold deck design, for use in a dual duct heating, ventilating, and air conditioning (HVAC) system. Heating of supply air is accomplished with steam heating coils within the hot deck of the air handlers, and cooling of supply air is accomplished with chilled water cooling coils located within the cold deck of the air handlers. The air handlers also include steam preheat coils to precondition incoming outside air and protect the cooling coils from freezing during the winter months.

New steam heating coils and cooling coils were installed in the late 1990s. During the 2008 site observations, it was noted that the existing steam heating coils in both the north and south Arch leg air handlers were in poor condition, as they are leaking and in need of replacement. The heating coils were replaced in 2010. The existing cooling coils in both the north and south air handlers are in fair condition. No leaks appear to be present in the cooling coils, and there are no problems or operational deficiencies known to park maintenance staff.

Very little archival information has been discovered with regard to the capacities of the existing cooling, heating, and preheat coils. Information gathered from an archival control schematic drawing indicates that the cooling

coils in both the north and south air handler appear to flow approximately 300 gallons per minute (gpm) of chilled water. This yields a total cooling capacity of approximately 1.5 million British thermal units per hour (Btu/h), or 125 tons of cooling capacity for each Arch leg air handler. Each cooling coil is served by a single three-way mixing valve.

The existing steam preheat coils for each Arch leg air handler are located within the return/outdoor air plenums. Each of the existing preheat coils is supplied by two heating control valves. Information obtained from an archival control schematic drawing indicates that the steam flow capacities of the preheat coil valves are 420 pounds of steam per hour (lb/h) and 200 lb/h, respectively. This gives the preheat coils in each air handler a total steam flow capacity of 620 lb/h. The heating capacity of the preheat coils in Btu/h is dependent upon the steam pressure utilized by the heating system.

There are two steam heating coils in each Arch leg air handler, each of which is supplied by two steam heating control valves. The steam flow capacities of the heating valves for each air handler's first heating coil are 1000lb/h and 666 lb/h, respectively, for a total steam flow capacity of 1,666 lb/h for the first heating coil in each air handler. The steam flow capacities of the heating valves for each air handler's second heating coil are 1,000 lb/h and 722 lb/h, respectively, for a total steam flow capacity of 1,722 lb/h for the second heating coil in each air handler. The total heating steam flow capacity for each air handler is thus 4,008 lb/h, including the preheat capacity. The heating capacity in Btu/h is again dependent upon the steam pressure utilized by the heating system.

Both the north and south Arch leg air handlers are equipped with a supply fan and a return fan. Park maintenance staff has indicated that

the fans are original to the air handler installation, and thus original to the construction of the Arch. The supply fans are large diameter centrifugal fans. The return fan type could not be verified, but the return fans are assumed also to be large diameter centrifugal fans. Fan blade type could not be physically verified during the investigation for either the supply or return fans; however, archival shop drawing documentation suggests that the fans are airfoil type. Motor horsepower for the supply and return fans could not be obtained during the investigation and no archival documentation has been found indicating the existing fan horsepower.

The supply fans are used for both hot duct and cold duct supply air distribution into the Arch legs and observation level. The return fans are used to provide return air back to the air handlers from the Arch legs, and to exhaust air from the return air stream, allowing for ventilation air to be brought into the system. Ventilation air appears to be ducted to each air handler from a large fresh air plenum in each mechanical room. The fresh air ductwork from each plenum leads to existing area wells located outside on the Arch grounds, from which the ventilation air for the Arch is brought into the system. The openings to these area wells are located at grade.

The supply fans for both the north and south Arch leg air handlers have been retrofitted with inverter-duty motors and variable frequency drives. This enables the air handlers to modulate the speed of the supply fans, and in so doing vary the amount of airflow being distributed from the hot and cold decks to the duct system. According to park maintenance staff, this modification was made circa 2003. No archival documentation has been found to confirm this information.

Both the supply and return fans in each air handler appear to be in fair condition. Close

inspection of the fans was not possible at the time of this investigation, and the condition assessment of the fans is based upon appearance, age, and information provided by park maintenance staff. The supply fan motors are in good condition, as they were installed concurrent to the addition of the supply fan variable frequency drives. The condition of the return fan motors is unknown, as the return fans could not be accessed or viewed during the investigation. No operational deficiencies in the return fans were reported by park maintenance staff, and it is assumed the return fan motors are in fair to good condition. Park maintenance staff has indicated that additional belts, sheaves, and pulleys are on hand for replacement of the fans and motors as needed.

The filter sections within the north and south Arch leg air handlers are located ahead of the supply fan sections, and appear to be in good condition. Pleated media filters within the filter section are replaced by park maintenance staff as required.

Arch Leg High Pressure Air Distribution System

The ductwork systems serving the Arch legs are original to the construction of the Arch and its associated mechanical systems. High pressure hot and cold supply air ductwork discharges from the north and south Arch leg air handler's respective hot and cold decks. This ductwork is then routed to and up through the north and south Arch legs to large mixing boxes located at various elevations within the legs (Figure 185). Each Arch leg currently contains nine mixing boxes. The first eight mixing boxes supply air directly into the Arch legs for conditioning within the legs themselves (Figure 186). The ninth mixing box in both the north and south legs is used to provide conditioned air into the observation level at the top of the Arch. The existing ductwork is supported as required at various points throughout the Arch legs by the Arch

structure itself and by the support structure for the trams. The existing ductwork is offset at various locations throughout the Arch legs as required by the varying Arch geometry, support structure, and mixing box locations.

Archival documents indicate that the high pressure hot and cold ductwork is double wall insulated construction from the point of connection at the air handler to a point above the lower tram load zones. The thickness and type of insulation is unknown, as it is not specified on the archival drawings. The documents do not indicate the ductwork in the Arch legs to be insulated, and no interior or exterior insulation exists on the ducts within the legs.

The high pressure hot and cold ductwork and associated support and anchorage appear to be in good condition throughout the Arch legs. No observable construction or operational deficiencies were observed with the ductwork or support systems.

The existing mixing boxes serving the Arch legs are large dual duct boxes with internal hot and cold plenums and mixing chambers (Figure 187). These mixing boxes are original to the construction of the Arch and its associated mechanical systems. Each mixing box connects to both the hot and cold high pressure supply air ductwork, and modulates the ratio of hot-to-cold airflow through the box in order to maintain a set point temperature.



Figure 185. Arch leg high pressure hot and cold supply ducts. Source: Alvine, 2008.



Figure 186. Arch leg mixing box. Source: Alvine, 2008.



Figure 187. Arch leg mixing box hot and cold duct connections. Source: Alvine, 2008.

Each existing box has eight outlets that supply conditioned air into the Arch legs: two outlets on both the top and bottom of the box, two outlets on the face of the box, and one outlet on either side of the box. The total combined airflow of the mixing boxes serving each leg of the Arch, excluding the mixing boxes and associated airflows serving the observation level, is 23,600 cfm in each leg. Not all of the boxes supply the same airflow throughout the Arch, and the method for determining the original design airflows at the box locations is unknown. No archival design data has been found to indicate the design methodology. Humidity has been noted as a problem within the Arch legs, and according to park maintenance staff the moisture levels reach points high enough to cause heavy condensation and even fogging within the Arch legs.

The dual duct mixing boxes serving the Arch legs appear to be in fair condition, due to the age and unknown internal condition of the mixing boxes. No operational deficiencies were observed or reported by park maintenance staff.

Observation Deck and Upper Tram Load Zone Air Distribution System

The observation level and upper tram load zones located at the top of the Arch are also served by the high pressure dual duct HVAC system serving the legs. The last or uppermost existing dual duct mixing box located in each leg is used to supply air to the observation level. These mixing boxes are original to the construction of the Arch and its associated mechanical systems. The hot and cold high pressure ductwork connects to these final mixing boxes in the same manner as it does to the mixing boxes serving the Arch legs themselves. The construction of these mixing boxes is different from that of the existing boxes serving the Arch legs, however.



Figure 188. Observation deck mixing box and duct connection. Source: Alvine, 2008.



Figure 189. Observation deck and upper tram load zone ductwork. Source: Alvine, 2008.



Figure 190. Observation deck supply air grille. Source: Alvine, 2008.



*Figure 191. Upper tram load zone air distribution.
Source: Alvine, 2008.*

The existing mixing boxes serving the north and south observation level do not have outlets located on the various sides of the mixing box enclosure. Instead, these boxes have a duct connection at the top side of the box, to which the ductwork serving the observation level is connected (Figure 188). According to archival shop drawing information, the existing mixing boxes serving the north and south observation level and upper tram load zones each have 2,500 cfm airflow capacity.

The observation level mixing boxes appear to be in fair condition, due to age and unknown internal condition. No operational deficiencies with the boxes were reported by park maintenance staff.

The existing ductwork serving the observation level leaves the respective north and south observation level mixing box as an approximately 20 inch round supply air duct, which is externally insulated with 1-1/2 inch fiberglass blanket insulation. This existing supply air duct runs between the existing mixing boxes and the north and south observation level tram load zones (Figure 189). Once the supply duct reaches the upper tram load zones, it penetrates the wall separating the Arch leg from the loading area,

and transitions to an approximately 36 inch wide by 4 inch tall, uninsulated supply air duct. This existing rectangular supply air duct then runs along the top of the observation level tram load zone, to the entryway of the observation level. At the north and south observation level entryways, an existing 36 inch wide by 4 inch tall duct mounted grille supplies airflow into the observation level (Figure 190). This arrangement is not original to the construction of the Arch. Park maintenance staff has indicated that the original observation level supply air system was abandoned in the mid 1990s, due to clogging of airflow openings, and lack of performance. No archival documentation could be found to confirm this timeframe, or to document exactly what modifications were implemented.

The upper tram load zones just below the observation level are served by existing linear slot diffusers in the walls opposite the tram doors. There are a total of nine diffusers in the wall of each tram load zone, and each diffuser is approximately 24 inches wide by 4 inches tall. The ductwork serving these existing diffusers is located behind the wall of the tram load zone (Figure 191).

The ductwork serving the observation level and upper tram load zones is in fair condition. The external insulation on the 20 inch round supply duct between the mixing box and upper tram load zone is torn in some locations. However no construction or operational deficiencies are apparent or were reported by park maintenance staff.

Lower Tram Load Zone Air Handlers and Mixing Boxes

There are currently two existing air handlers that serve the lower tram load zones at the base of the Arch legs: one unit serving the north load zone and one serving the south load zone (Figure 192). These units are not original

to the construction of the Arch, as they were designed and installed circa 1998. In addition to the non-original air handling units currently serving the tram load zones, there are also two existing non-original mixing boxes that were installed concurrent with the air handlers, one serving the north loading area and one serving the south. The existing mixing boxes replaced the original mixing boxes and are fed by the existing high pressure dual duct system and Arch leg air handlers.

The existing north and south lower tram load zone air handlers deliver 4,800 cubic feet per minute (cfm) of airflow each to the respective lower tram load zone via a 24 inch round supply air duct and large supply air linear slot diffuser. Also typical for both the north and south systems, air is returned back to the air handlers from the lower tram load zones through large return grilles in the walls. The air handlers are direct expansion (DX) type, cooling only units, which are hung from the ceiling structure of the tram load zones in a horizontal configuration. Each air handler has an associated condensing unit that is located within the fresh air chase adjacent to the tram load zones. The existing condensing units are each 10 tons total cooling capacity. Condenser air is discharged upward and out of the fresh air chase via ductwork connected to the units. Both the north and south tram load zone air handlers and condensing units are manufactured by Engineered Air.

The existing air handlers serving both the north and south tram load zones appear to be in good condition. No operational deficiencies with the air handlers were reported by park maintenance staff. The associated condensing units could not be accessed or viewed during the investigation. The condition of these condensing units is assumed to be fair to good based on the age of the units, and no operational deficiencies were noted by park maintenance staff.



Figure 192. Lower tram load zone air handler and piping connection. Source: Alvine, 2008.

Both the existing north and south lower tram load zone mixing boxes are served by 16 inch round hot and cold ducts that connect to existing 12 inch round high pressure hot and cold ducts from the Arch leg ductwork. The mixing boxes each supply 3,600 cfm of airflow to the tram load zone via ductwork and ceiling and wall-mounted linear slot diffusers. This air then either filters back to the return air chase/Arch leg by whatever means of available air path exists, or spills out to the visitor center/museum. The existing mixing boxes are manufactured by Titus.

The existing mixing boxes serving both the north and south tram load zones appear to be in good condition. No operational deficiencies with the mixing boxes were reported by park maintenance staff.

Chillers and Cooling Tower

The existing north and south Arch leg air handlers, as well as other air handlers serving the visitor center/museum and ancillary spaces, currently utilize chilled water as their cooling source. This chilled water is made by two existing centrifugal type chillers located in the north mechanical room. The existing chillers are manufactured by York, and are not original to the construction of the Arch and its associated mechanical systems (Figure 193).



Figure 193. Existing 300 ton chiller. Source: Alvine, 2008.

According to park maintenance staff, the existing chillers were installed circa 1998. No archival documentation has been found to confirm this timeframe. Design documents were located in the maintenance office; however, those drawings documented the design for removal of the original centrifugal chiller and installation of two new chillers. The drawings are dated 1979, so it is possible that the chiller system has been updated twice. No archival documentation could be found to support this supposition, however, and the 1979 chiller replacement drawings were not included in the scanned archival information.

Each existing chiller provides approximately 300 tons of cooling capacity. The chillers operate with R-123 refrigerant and include integral variable frequency drives for capacity control. The current configuration of the existing chillers is such that they operate in series, each providing approximately 50 percent of the design cooling capacity, and staging as required.

The existing chillers are in good condition and have been well maintained by the park maintenance staff. No operational problems or deficiencies were observed or reported with the existing chillers.



Figure 194. Baltimore Air Coil cooling tower. Source: Alvine, 2008.

The existing chillers are water cooled, with condenser water cooled by an existing Baltimore Air Coil cooling tower (Figure 194). The cooling tower is located outside, adjacent to the generator building to the northwest of the north mechanical room. The existing cooling tower is not original to the construction of the Arch or its associated mechanical systems, and according to park maintenance staff was installed in 2007. No archival documentation has been found to confirm this timeframe. The existing cooling tower has a capacity of approximately 680 tons. The two existing cooling tower fans are each 40 horsepower and incorporate variable frequency drives. This enables the fans to be reduced in speed as required to match condenser water load requirements. The cooling tower also incorporates the use of sump heaters, which enable operation of the unit in low ambient air temperatures without freezing the water in the sump basin. Netting has been placed around the base of the cooling tower to help prevent leaves and debris from being introduced into the airstream, thus reducing degradation of the unit capacity and nuisance maintenance problems.

The existing cooling tower is in good condition and is well maintained by park maintenance staff. No operational deficiencies

were observed or reported with the cooling tower.

Chilled and Condenser Water Pumps and Piping System

The existing chilled water serving the Arch mechanical systems is piped in a primary-secondary piping system configuration. There are three existing pumps that serve the chilled water system associated with the Arch. These pumps are located in the north mechanical room. The first pump is a primary chilled water pump, which provides chilled water flow through the existing chillers. This pump is a 20 horsepower end suction pump, manufactured by Taco (Figure 195). The second and third pumps are secondary chilled water pumps, which provide chilled water flow to the air handling units and associated cooling coils. These are 50 hp split case double suction pumps, also manufactured by Taco. The existing chilled water pumps are not original to the construction of the Arch and its associated mechanical systems. No archival documentation could be found regarding the timeframe for the pump replacements, but it is assumed that the existing pumps were installed to replace the previous ones when the existing chillers were installed, circa 1998.

The primary chilled water pump does not incorporate a variable frequency drive, and thus is a constant volume pump. Both of the secondary chilled water pumps are on variable frequency drives, and the secondary chilled water distribution system is therefore a variable volume system. Again, no archival documentation has been found to indicate when the variable frequency drives were installed, but it is assumed they were installed concurrent with the chiller and possible pump replacement, circa 1998.

The existing chilled water pumps and pump motors are in good condition. It is unknown

whether the existing pump motors are original to the pumps themselves, and no archival documentation has been found to confirm a timeframe for pump motor replacement.

The majority of the chilled water piping serving the Arch chilled water system is original to the construction of the Arch and associated mechanical systems. The chilled water piping is steel pipe, with welded joints and fittings. The original chilled water piping was likely altered as needed at the time of the chiller replacement, as the chiller system was modified from a single chiller to a two chiller system. No archival documentation has been found indicating the original chilled water piping layout or the scope of subsequent piping alterations, however.

The existing chilled water piping system is in fair condition. No leaks or major operational deficiencies were observed or reported by park maintenance staff. However, portions of the existing chilled water piping system within the north mechanical room are missing insulation. The chilled water piping insulation that is present in the mechanical room appears to be a mix of old insulation and new insulation. Some of the existing chilled water piping insulation in the north mechanical room has been tagged as possibly containing asbestos (Figure 196). The chilled water piping insulation as a whole is in poor condition. The age of the piping system, unknown condition of the piping interior, condition of the piping insulation, and unknown history of water quality control contribute to the assessment of the piping. Some degree of internal piping deterioration or erosion may also be present.



Figure 195. Existing chilled water pump. Source: Alvine, 2008.



Figure 196. Asbestos tag on existing chilled water piping. Source: Alvine, 2008.



Figure 197. Condensing water pumps. Source: Alvine, 2008.

The existing condenser water system serving the chillers and cooling tower consists of three condenser water pumps and the existing piping system. The existing condenser water pumps are identical to one another. They are 25 horsepower end suction pumps, and are manufactured by Taco (Figure 197). The existing condenser water pumps are not original to the construction of the Arch or its associated mechanical systems. No archival documentation has been found to confirm the timeframe of the condenser water pump replacement, but it is assumed the pumps were installed concurrent with the existing chillers and chilled water pumps, circa 1998.

The pumps do not incorporate variable frequency drives, and are thus constant volume pumps. Two of the three pumps operate simultaneously in a parallel pumping configuration, each providing half of the required condenser water flow rate. The third pump is a standby pump. The pump operation is alternated as required to maintain equal runtime on all three of the existing pumps.

The existing condenser water pumps and pump motors are in good condition. According to park maintenance staff, the pumps do tend to cavitate slightly when the pump suction strainers become clogged with debris, which is typical for end suction pumps. It is unknown as to whether or not the existing pump motors are original to the pumps themselves, and no archival documentation has been found to confirm a timeframe for pump motor replacement.

The existing condenser water piping system serving the Arch chillers and cooling tower consists of grooved end steel piping with mechanical joints and fittings. This piping system is not original to the construction of the Arch and its associated mechanical systems. No archival documentation has been found to confirm the timeframe for the

installation of the existing condenser water piping system. It is assumed that the existing condenser water piping system was installed concurrently with the existing cooling tower, circa 2007.

The existing condenser water piping is in good condition. No operational deficiencies were observed or reported by park maintenance staff.

Steam and Condensate Piping System

Central plant steam for heating is currently purchased from Tri-Gen energy. The existing steam and condensate piping serving the north and south Arch leg air handling unit heating coils appears to be in large part original to the construction of the Arch. Associated steam valves and condensate traps that could be viewed also appear to be largely original to the Arch and its original mechanical systems. The existing steam and condensate piping is welded steel pipe. The steam and condensate piping connections to the existing Arch leg air handling units heating coils could not be observed, due to a lack of accessibility. Condensate pumps located in a pit in the north mechanical room return the condensate back to the central plant condensate return mains which enter the north mechanical room.

Park maintenance staff indicated that the existing steam and condensate piping rarely experiences leaks, and is repaired on an as-needed basis when it does. The steam and condensate piping is thus in fair condition, largely due to age and unknown interior conditions. The existing insulation on the steam and condensate piping is in poor condition, as it is missing or compromised in several locations within the north and south mechanical rooms. In addition, the existing steam and condensate piping insulation has been noted to possibly contain asbestos.

Temperature Controls

The existing temperature controls system serving the Arch and associated mechanical systems is a combination of pneumatic controls and direct digital controls (DDC). The existing pneumatic controls and control devices are original to the construction of the Arch and its associated mechanical systems; the existing digital controls are not original to the Arch construction. According to Mike Hirons at Eagle Energy in St. Louis, digital control components were introduced into the Arch HVAC systems in 1984. The controls components installed at that time are unknown. According to Mr. Hirons, the original digital control system installed in 1984 has been upgraded to include additional HVAC components, as well as security/access systems. The existing digital control system is manufactured by Andover. No archival documentation could be found to confirm the manufacturer of the existing original pneumatic controls.

Little archival information has been found to indicate the extent of the Andover digital control system. Park maintenance staff has indicated that the existing Andover system controls most of the functions with regard to the systems and components serving the Arch. Where original pneumatic control devices are still utilized, pneumatic-to-digital control transducers have been installed to allow the digital control system to recognize pneumatic control signals, and operate the devices as required. According to park maintenance staff, the existing air compressor serving the existing pneumatic controls was installed circa 2007 to replace the previous air compressor. It is unknown whether the previous air compressor was original to the Arch construction. The existing pneumatic controls air compressor is located in the south Arch mechanical room. The existing master pneumatic control panel is located in the north Arch mechanical room. A majority of the

functions originally served by the master pneumatic control panel have been switched over to the existing digital control system. However, according to park maintenance staff, the pneumatic control panel does maintain some functionality, including status and monitoring. A comprehensive points list for all of the existing control points associated with both the pneumatic and digital control systems has not been found within the archival documentation.

The existing temperature controls system is in fair condition overall, due mostly to the age of the system and the non-uniform nature of the system and components. No operational deficiencies were observed or reported by park maintenance staff. Condition of individual control components of both the pneumatic and digital systems is fair, due mostly in part to the age of the components.

Electrical

Primary Power Distribution

The primary electrical power distribution system, including the pad-mount transformers, is owned and maintained by the local power company, AmerenUE. The pad-mount transformers are located in an open-top with grate in-ground transformer vault (Figure 198). The primary power distribution system is in good condition. There are three electrical kilo-watt-hour meters labeled “Arch Main,” “Bi-State Main,” and “Site Lighting” (Figures 199 and 200).

Code Compliance

The primary electrical power distribution system, including pad-mount transformers, appears to comply with code regulations.

Life Expectancy

The primary electrical power distribution system will be maintained by the local power company. The life expectancy of the current primary electrical distribution system is unknown.



Figure 198. The pad-mount transformers are located in an open-top with grate in-ground transformer vault. Source: Alvine, 2008.



Figure 199. Two of the meters, labeled “Arch Main” and “Bi-State Main.” Source: Alvine, 2008.



Figure 200. The third meter, labeled “Site Lighting.” Source: Alvine, 2008.

Electrical Service

The building has a Culter Hammer Power line “C” 480Y/277-volt, three phase, four wire switchboard with two 2,000 amp main breakers and two split buses connected via automatic transfer switches (Figure 201) to the emergency distribution system (Figure 202 through 204).

The switchboard has mimic bussing on the face of the switchboard (Figure 205). Each of the two 2,000 amp main breakers is connected to a power company pad-mount transformer via a 2,000 amp electrical feeder. The overcurrent protective devices in the switchboard are fusible switches (Figure 206 and 207) and circuit breakers (Figure 208). The switchboard has a split bus, which separates normal and emergency power. The switchboard was installed in December 1998 after an electrical fire destroyed the original switchboard. There is a separate outside electrical service for site lighting.

At the top of the Arch below the visitor observation deck for each leg there is a lighting system enclosed breaker, a power system enclosed breaker, and a tie breaker. The tie breakers connect power to each side of the Arch.

The electrical service system is in good condition.

Distribution of electrical power downstream of the electrical service switchboard consists of panelboards with circuit breaker overcurrent devices (Figure 209). Dry type transformers are used to transform the voltage from 480 volts to 208Y/120 volts (Figure 210). In the Arch legs there are several specially made combination 480-208Y/120 volt transformers with 208Y/120 volt panelboards. The transformer is wedge shaped to fit in the corner of the Arch legs. The downstream distribution system is in good condition.

The manufacturer of the existing power distribution equipment in the arch is Federal Pacific Electric Company.

Code Compliance

The 480Y/277 volt, three phase, four wire, 2,000 amp switchboard itself complies with code requirements except for required working clearance. The existing temperature control gauge cabinet extends into the switchboard work clearance area, which is a violation of the National Electrical Code.

Life Expectancy

The life expectancy of the electrical service, and downstream panelboards and transformers should exceed twenty years.

Existing Drawing Reference

Arch One Line Diagram June 2007
(Figure 211).



Figure 201. Automatic transfer switches in switchgear. Source: Alvine, 2008.



Figure 204. Main electrical switchboard in mechanical room. Source: Alvine, 2008.



Figure 202. Main electrical switchboard in mechanical room. Source: Alvine, 2008.



Figure 205. Main electrical switchboard in mechanical room. Source: Alvine, 2008.



Figure 203. Main electrical switchboard in mechanical room. Source: Alvine, 2008.

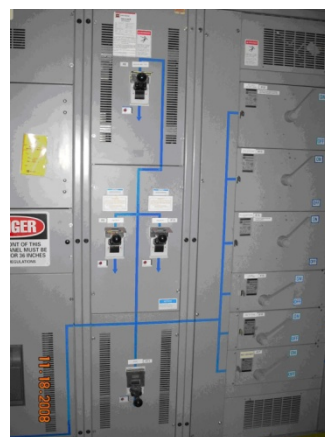


Figure 206. Main electrical switchboard in mechanical room. Source: Alvine, 2008.



Figure 207. Main switchboard section with fusible switches. Source: Alvine, 2008.



Figure 210. Electrical panels in mechanical room. Source: Alvine, 2008.



Figure 208. Main electrical switchboard in mechanical room. Source: Alvine, 2008.

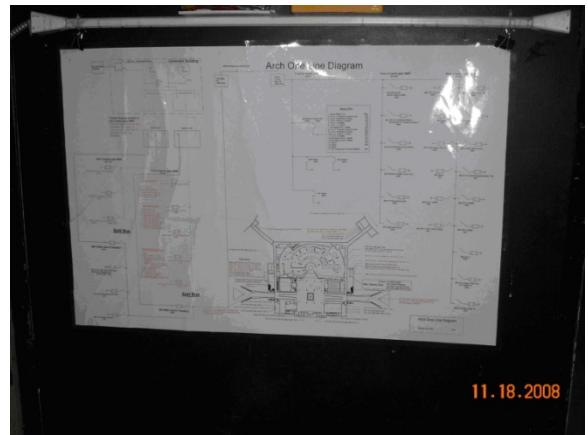


Figure 211. Electrical one-line diagram. Source: Alvine, 2008.



Figure 209. Electrical panels in mechanical room. Source: Alvine, 2008.



Figure 212. Electrical panels in mechanical room. Source: Alvine, 2008.

Emergency Power Distribution

In 2009 the original 235 KW and 300 KW diesel powered emergency generators (Figure 213) were replaced by two new 350 KW diesel powered emergency generators.

During the transitional period between removing the two original emergency generators and installing the two new emergency mounted generators were on site with temporary power connections back to the emergency power distribution system (Figure 214).

The two 350 KW emergency generators are located in a separate building (Figures 215 through 217).

There are two automatic transfer switches in the main 480Y/277 volt, 2,000 amp switchgear in the mechanical room. There is an outside load bank with three stages of resistance: 120 KW, 180 KW, and 240 KW (Figure 218).

The two new 350 KW generators and the emergency power distribution system are in good condition.

Code Compliance

The emergency power distribution system with the two new 350 KW generators, existing automatic transfer switches, emergency feeders, emergency panels, and connected loads meet current life safety code compliance issues.

Life Expectancy

The life expectancy of the emergency distribution system should exceed twenty years. The two 350 KW emergency generators will require periodic maintenance.



Figure 213. Emergency generator and control panel. Source: Alvine, 2008.



Figure 214. Emergency generator building. Source: Alvine, 2008.

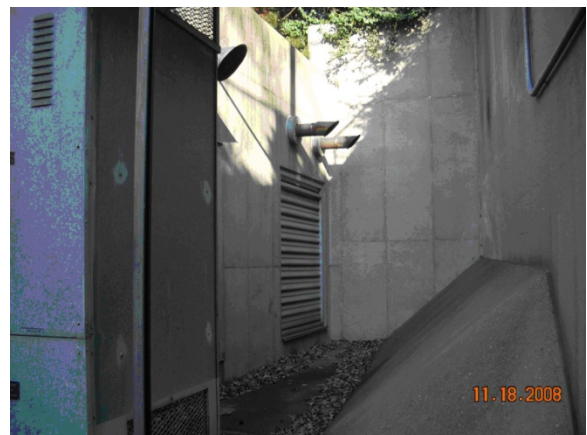


Figure 215. Emergency generator exhaust pipes and ventilation louver. Source: Alvine, 2008.



Figure 216. The outside temporary emergency engine generators. Source: Alvine, 2008.



Figure 217. Generator load bank. Source: Alvine, 2008.



Figure 218. Automatic transfer switches in switchgear. Source: Alvine, 2008.

Existing Drawing Reference

- Arch Electrical Distribution One Line diagram June 2007
- 366/41043A August 31, 1977
- Drawings E-1, E-5, E-6, E-7, and E-8 show existing generator building.

Interior Wiring

The electrical wiring in the Arch proper consists of conductors installed in threaded steel conduit (Figure 219 and 220). Electrical wiring in mechanical rooms consists of conductors installed in electrical metallic tubing (EMT). The EMT conduits have a mixture of set screw fittings and compression fittings. Flexible steel conduit is used in tight bend areas (Figure 221). The routing of the conduits in the Arch legs is in the corner of the legs. Special triangular shaped pull-boxes are used. The copper wire insulation used during the original construction period was probably type THW or TW. The tram load zones, which were remodeled in 1998, probably have copper wire conductors with THHN/THWN insulation. The original interior wiring is in fair condition due to its age. The 1998 wiring is in good condition.

Code Compliance

Due to the length of conduit runs in the Arch proper, expansion/contraction fittings should probably have been used on some runs of conduits. No expansion contraction fittings were observed. Routing and support of conduits comply with current code requirements.

Life Expectancy

The life expectancy of the original interior wiring should exceed ten years and will need to be upgraded sometime in the future. The tram load zones 1998 interior wiring should have a life expectancy of at least thirty years.

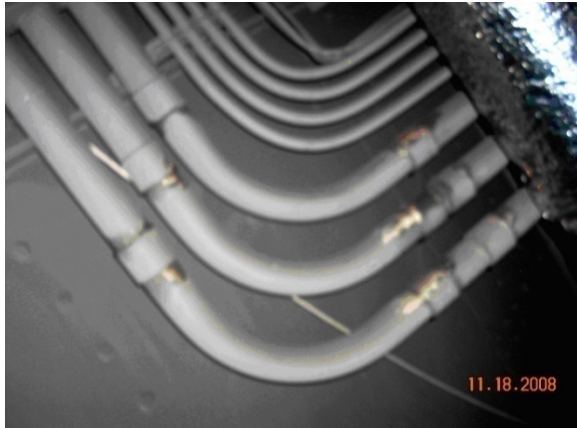


Figure 219. A conduit rack in Arch leg. Source: Alvine, 2008.



Figure 220. Threaded rigid conduit in subfloor area below visitor platform. Source: Alvine, 2008.



Figure 221. Flexible conduits to equipment in subfloor area below visitor platform. Source: Alvine, 2008.

Existing Drawing Reference
NHS – JNEW-3078

Grounding

The grounding electrode system connection from the main switchboard consists of connections to the water main and building steel. The equipment ground system for feeders and branch circuits utilizes the conduit system as a grounding path. The grounding system is in good condition.

Code Compliance

The grounding electrode system and equipment grounding meet the minimum National Electrical Code requirements for grounding.

Life Expectancy

The grounding electrode system has a long life expectancy. The life expectancy of the equipment grounding system is dependent upon the conduit connection integrity and its ability to act as a grounding path.

Lightning Protection

At the top of the Arch proper there are lightning arrestors with cables routed in the legs of the Arch that extend down to the concrete footings. According to park staff the lightning protection system works adequately. The lightning system is in fair condition due to its age.

Code Compliance

The lightning protection system at the Arch comply with the Lightning Protection Institute requirements for a Class II system.

Life Expectancy

The life expectancy of the lightning protection system is unknown.

Lighting

The legs of the Arch proper along the stairs have a mixture of incandescent “fruit jar” type lighting fixtures (Figures 222 and 223) and recently added halogen flood lighting fixtures. Some of the halogen flood lighting fixtures have individual control switches. The lighting is connected to the emergency power distribution system. The lighting at the top of the Arch has wall-mounted or under-railing fluorescent lighting fixtures (Figures 224 through 226). The lighting fixtures in the tram equipment area below the observation deck are a strip fluorescent (Figure 227). The tram load zones, which were remodeled in 1998, have incandescent downlights with self ballasted fluorescent lamps, wall washer lighting fixtures with T-5 fluorescent lamps, track mounted lighting fixtures with incandescent PAR lamps, and track mounted lighting fixtures with halogen lamps (Figures 228 through 230). There is an aircraft warning light located on the top of the Arch. The lighting system is in fair condition.

Code Compliance

Since the Arch is a historic structure, the lighting system is not required to comply with the IECC 2003 or ASHREA 90.1 Energy Conservation codes.

Life Expectancy

The life expectancy of the lighting fixtures should exceed ten years with normal periodic lamp replacement.

Existing Drawing Reference

Drawing number NHS-JNEW-3078 shows lighting fixtures in the Arch legs.

The under railing lighting fixtures at the top of the Arch are detailed on section A-A and B-B, Sheet 7, Job Number M580, Sachs Electrical Drawings, Dated 1-4-64.



Figure 222. Incandescent lighting fixture along stairway in Arch leg. Source: Alvine, 2008.



Figure 223. Incandescent lighting fixtures in Arch leg. Source: Alvine, 2008.



Figure 224. Under railing fluorescent lighting at top of Arch. Source: Alvine, 2008.



Figure 225. Lighted sign at top of Arch. Source: Alvine, 2008.



Figure 228. Lighting in the tram load zone. Source: Alvine, 2008.



Figure 226. Fluorescent lighting fixture at observation level. Source: Alvine, 2008.



Figure 229. Track lighting and down lighting in tram load zone. Source: Alvine, 2008.

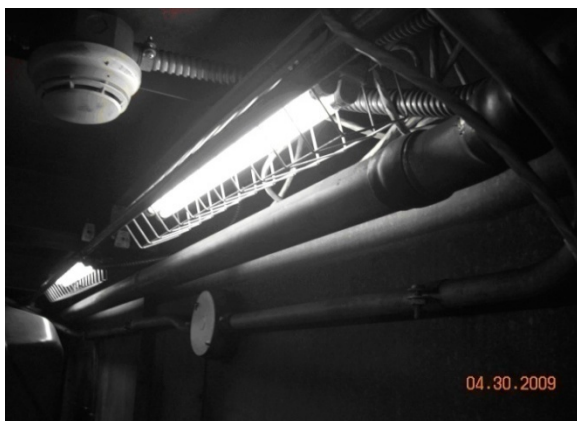


Figure 227. Fluorescent lighting fixture and fire alarm smoke detector in subfloor area below visitor platform. Source: Alvine, 2008.



Figure 230. Track lighting in the tram loading area. Source: Alvine, 2008.



Figure 231. Quartz flood lighting fixture and switch in Arch leg. Source: Alvine, 2008.



Figure 232. Convenience 120-volt, 15-amp receptacle along stairwell in Arch leg. Source: Alvine, 2008.



Figure 233. Lighting fixture switch along stairwell in Arch leg. Source: Alvine, 2008.

Devices

In the Arch legs there are 120 volt, 15 amp, grounded ivory colored receptacles mounted along the side of the stairwells (Figure 232). Switches alongside the stairwells control individual halogen floodlights (Figure 233). There are 120 volt, 15 amp, grounded convenience receptacles on the observation level and at the tram load zones. The devices are in fair condition due to age and accumulation of oily graphite.

Code Compliance

The devices comply with minimum National Electrical Code requirements.

Life Expectancy

The life expectancy of the devices in the Arch legs should exceed five years. Devices in other locations should exceed ten years.

Existing Drawing Reference

Drawing Number NHS-JNEW-3078 shows devices in the Arch legs.

Exit Lighting

The original exit lighting fixtures have red letters and are illuminated with incandescent lamps (Figure 234). The tram load zones have exit lighting fixtures with light-emitting diode (LED) or fluorescent lamps (Figure 235). The exit lighting fixtures are located in the lower tram load zones, upper tram load zones, at the top of the Arch in the visitor observation area, and back of house in the corridors. The original exit lights are in fair condition, as they do not use highly energy efficient lamps. The tram load zone exit lighting fixtures are in good condition.

Code Compliance

The exit lighting type, placement, and quantity meet current life safety codes.

Life Expectancy

The life expectancy of the original exit lighting fixtures should exceed ten years, except for lamp replacement. The tram load zone exit lighting fixtures should have a life expectancy of twenty years.



Figure 234. Exit lighting fixture at observation deck level. Source: Alvine, 2008.



Figure 235. Exit light and fire alarm/horn strobe light in tram load zone. Source: Alvine, 2008.

Emergency Lighting

The battery powered self contained unit emergency lighting fixtures have integral batteries and are the car headlight type. The emergency lights are located in the lower tram load zones, upper tram load zones, and at the top of the Arch in the visitor observation platform (Figures 236 through 238). The normal use lighting fixtures in the Arch legs, tram load zones, and visitor viewing area at the top of the Arch are also connected to the emergency generator distribution system. The emergency lighting fixture type, placement, and quantity appear to meet life safety codes.

Code Compliance

The emergency lighting system complies with Life Safety Code requirements.

Life Expectancy

The battery powered self contained unit emergency lighting fixtures have a life expectancy of approximately ten years from time of installation before batteries need replacement.



Figure 236. Emergency battery pack lighting at tram load zone at top of the Arch. Source: Alvine, 2008.



Figure 237. Emergency battery pack lighting in tram load zone. Source: Alvine, 2008.



Figure 238. Exit light emergency battery pack light, and fire alarm horn/strobe light. Source: Alvine, 2008.

Fire Alarm System

The fire alarm system was updated in 2007. The new fire alarm system is a Simplex Model No. 4100V Voice Evacuation type. The main fire alarm control panel is located in the maintenance office (Figure 239). Manual pull stations are located in the upper tram load zones, at the top of the Arch in the visitor platform area, and back of house in corridors and mechanical rooms. Combination fire alarm speaker strobes are located in the upper tram load zones, lower tram load zones, at the top of the Arch in the visitor platform area, and back of house in corridors and mechanical rooms (Figures 240 and 241).

There are no fire alarm smoke detectors in the Arch legs. There are no fire alarm strobe lights or speakers in the Arch legs.

There are three node panels: one in the maintenance office, one in the south leg, and one in the courthouse. The fire alarm system is in good condition. The fire alarm system includes the following Simplex fire alarm equipment and catalog numbers:

- 4100-9111 Addressable Fire Alarm Control Panel Node 4009-9201
- Addressable Fire Alarm Control Panel Node 4009-9201
- Pull Station. “P” subscript indicates device has STI-1130 protective cover
- 4098-9792 Addressable Truealarm Smoke Sensor Base with 4098-9714
- Photoelectric Truealarm Smoke Sensor. “B” subscript indicates device will be black in color
- 4098-0792 Addressable Truealarm Smoke Sensor Base with 4098-9733 Truealarm Heat Sensor
- 4098-9758 Addressable Truealarm Duct Smoke Sensor Housing with Programmable Relay Output; 4098-9714 Photoelectric Truealarm Smoke Sensor; 2098-9797 Sampling Tube; and 4098-

- 9843 (PAM-SD) Encapsulated Control Relay Sprinkler Water Flow Switch
- Sprinkler Tamper Switch
- 2088-9607 Door Holder
- 4090-9001 Addressable Supervised Monitor IAM with 4090-9810 and 4090-9807 Surface Cover Plate
- 4090-9002 Addressable Relay IAM with 4090-9802 Surface Cover Plate
- D296 Beam Smoke Detector Transmitter
- D296 Beam Smoke Detector Receiver
- 4090-9101 4-Wire Monitor ZAM with 4090-9802 Surface Cover Plate
- 4906-9151 Truealert Multi-Candela Speaker/Visible, Wall Mounted, subscript denotes candela rating and speaker tap
- 4906-9154 Truealert Multi-Candela Speaker/Visible Ceiling Mounted, subscript denotes candela rating and speaker tap
- 4906-9101 Truealert Multi-Candela Visible “Only” Device, wall mounted, subscript denotes candela rating
- 4906-9104 Truealert Multi-Candela Visible “Only” Device, ceiling mounted, subscript denotes candela rating
- VT-157UCR, 4 inch square UL Listed Loudspeaker, Surface Mounted On WBB-R Weather-Resistant Back Box, subscript denotes speaker tap
- UHT70C-U51-8, 8 inch round 10W UL Listed Loudspeaker, Ceiling Mounted, with 95-8 Enclosure, subscript denotes speaker tap

Code Compliance

The fire alarm system meets all current Life Safety Code requirements.

Life Expectancy

The fire alarm system should have a life expectancy of at least twenty years.



Figure 239. Main fire alarm control panel. Source: Alvine, 2008.



Figure 240. Fire alarm horn/strobe light at observation deck level. Source: Alvine, 2008.



Figure 241. Fire alarm horn/strobe light. Source: Alvine, 2008.

Intrusion Alarm System

The Arch entrance has intrusion alarm motion detectors (Figure 242). There are no intrusion detection alarm devices in the Arch legs, tram load zones, or observation level area. Access doors have card access key pads. The intrusion alarm system is in good condition.

Code Compliance

There are no codes that cover intrusion alarm installations.

Life Expectancy

The life expectancy of the intrusion alarm motion detectors should be at least twenty years.



Figure 242. Security camera. Source: Alvine, 2008.

Communication System

At the top of the Arch in the visitor observation area, there are telephone and communication speakers (Figures 243 through 245). The paging system is through the fire alarm speakers. The paging zones are the lobby, tram load zones, and museum. The communication system is judged to be in fair condition due to the lack of speakers in the Arch legs.

Life Expectancy

The life expectancy of the communication system should exceed twenty years.

Existing Drawing Reference

366/60,067 June 9, 2006 EO



Figure 243. Communication system at observation deck level. Source: Alvine, 2008.



Figure 244. Communication speaker at observation deck level. Source: Alvine, 2008.



Figure 245. Communication cabinet in tram load zone. Source: Alvine, 2008.

Mechanical Equipment Connections

The main mechanical room air handling units are powered through variable frequency drive controllers (Figures 246 through 248). The pumps are powered through motor starters or variable frequency drive controllers (Figures 249 and 250). The motor feeders are routed to the HVAC motors in rigid metallic conduit. The final connections to the motors are made with flexible metal conduits. The electrical connections to mechanical equipment are in good condition.

Code Compliance

The electrical wiring to mechanical equipment meets the standards of the National Electrical code.

Life Expectancy

The life expectancy of the electrical wiring to mechanical equipment should exceed twenty years.

Existing Drawing Reference

366/80002 September 12, 1979

Drawings: E-1, E-2, E-3, E-4, E-5, E-6 and E-7



Figure 246. Original mechanical equipment control cabinet. Source: Alvine, 2008.



Figure 249. Variable frequency drive controllers in mechanical room. Source: Alvine, 2008.



Figure 247. Air handling unit variable frequency drive controllers. Source: Alvine, 2008.



Figure 250. Electrical combination motor starter disconnect in mechanical room. Source: Alvine, 2008.



Figure 248. Air handling unit variable frequency drive controller. Source: Alvine, 2008.

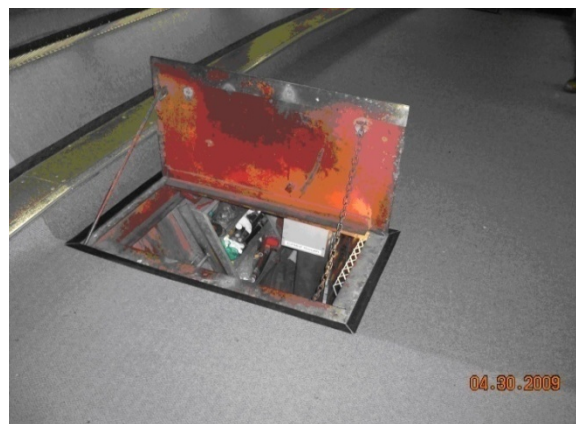


Figure 251. Access door to subfloor area below visitor platform. Source: Alvine, 2008.

Tram Equipment Connections

The tram equipment controllers are located at the top of the Arch visitor viewing area below the floor, with access through floor hatches (Figure 251). There are two, 125 horsepower, 75 KW motor generator sets and two, 125 horsepower traction hoists at the top of the Arch visitor viewing area below the floor. The area below the floor at the top of the Arch has four 200 amp three pole disconnect switches and four 30 amp three pole disconnect switches that serve motor generator sets and tram controllers. There are 480-240 volt, three phase transformers in the area below the floor at the top of the Arch, which power the trolley bus bars. The electrical feeders and branch circuits in this area are installed in rigid galvanized steel conduits. The electrical connections to the tram equipment are judged to be in fair condition due to age and working space that is insufficient to comply with current codes (Figures 252 through 254).

Code Compliance

Due to the lack of insufficient working space in front of all electrical equipment and insufficient head clearance, the area does not meet current National Electrical Code requirements.

Life Expectancy

Life expectancy of the electrical connections to the tram equipment should exceed ten years.



Figure 252. Fluorescent lighting fixture and fire alarm detector in subfloor area below visitor platform. Source: Alvine, 2008.

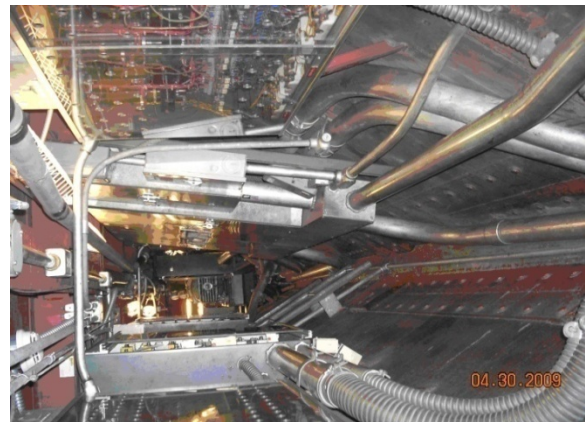


Figure 253. Tram electrical controls in subfloor area below visitor platform. Source: Alvine, 2008.



Figure 246. Threaded rigid conduit in subfloor area below visitor platform. Source: Alvine, 2008.

Plumbing

The existing plumbing system serving the Arch and lower tram load zones is limited to the sump pumps located at the lower tram load zones. These existing sump pumps serve to remove groundwater, which seeps up from the lower tram load zones due to the high water table at the Arch site (Figure 255). Close inspection of these sump pumps was not possible at the time of the field investigation. The existing sump pumps are column type pumps, with motors mounted above the sump basin and the pump assembly located in the sump basin. It is unknown as to whether or not the current sump pumps are original to the construction of the Arch. No archival documentation has been found to confirm replacement of these sump pumps. Park maintenance staff has indicated that the sump pumps have been rebuilt in the past, and that the pump motors have been replaced within the last five years.

There is one sump pit in each Arch leg at the tram load zones. Each of these sump pits contains two sump pumps, with a total of four sump pumps serving the Arch and tram load zones. It is assumed that a single pump at each pit can accommodate the full incoming water flow, and that the second pump in each pit is a fully redundant pump. No archival documentation has been found indicating the original design capacity of the pumps or the termination point for the sump pump discharge. The existing sump pits that contain the pumps did not appear to be sealed pits. Water can be heard entering the sump pits at a relatively high rate, thus the pumps are required to run often.

The sump pumps are in good condition, having been recently rebuilt. No operational deficiencies were observed during the investigation or reported by park maintenance staff.



Figure 255. Arch leg sump pumps. Source: Alvine, 2008.

Fire Suppression

The existing fire suppression system serving the Arch is limited to branches of the existing wet pipe sprinkler system that serve the lower tram loading areas. No archival documentation has been found that indicates the extent of the existing fire sprinkler piping system, which is original to the construction of the Arch.

Archival documentation indicates that the original fire sprinkler system serving the visitors center was revised and expanded in 1994 to further serve the visitors center expansions. The fire sprinkler system was again expanded to provide coverage for the north and south lower tram loading areas in 1998. No operational or coverage deficiencies were apparent within the north and south lower tram load zones. No dedicated zone valves or flow switches for the tram load zones were indicated in the archival documentation reviewed or observed during the field investigation.

The north and south legs of the Arch, as well as the observation level, are not currently addressed or protected by any fire suppression system.

The existing fire suppression system serving the north and south lower tram loading areas appears to be in good condition.

Code Compliance Review

Previous Studies

Several previous studies conducted on the Gateway Arch, including a limited accessibility review, were made available to the Historic Structure Report team for review. Specific reports including studies on tram rehabilitation, security, and mechanical/electrical system improvements. A previous accessibility study conducted by personnel from the National Transportation Board apparently was never published and/or copies have not been found in the JNEM archives. Additional studies have subsequently been developed on achieving accessible routes through the site to the Gateway Arch and visitor center. One such effort was slated for implementation but has not been funded or advanced due to concerns about a negative impact upon historic character and fabric. Refer to Appendix F for a copy of the project management information system (PMIS) statement regarding efforts to address accessibility. The NPS has recently adopted a new General Management Plan (GMP), which contains a Preferred Alternative that addresses accessibility. A design competition to improve the accessibility of both JNEM park grounds and accessibility to the Gateway Arch complex has been proposed. The goal of the design competition would be to address the challenges of providing accessibility while minimizing impacts to the character-defining features of the historic design.

Code Assessment

The 2006 edition of the International Building Code (IBC) and the 2000 edition of the National Fire Protection Association (NFPA) 101 Life Safety Code have been used as the basis of the review of existing conditions to identify parts of the structure that do not comply with current building code and life safety design requirements.

The focus of this study is specific to the Gateway Arch itself, including the trams, stairs, and observation level located at the top of the Arch structure. The study also includes the pedestrian access path starting just outside of the main entrances to the Arch and extending into the visitor center, and leading to the tram load zones located at the base of each leg of the Arch. This study does not include the ticketing area or any of the other portions of the visitor center or the Museum of Westward Expansion.

The *International Building Code*, Chapter 34, states that the provisions of the IBC relating to the construction, repair, alteration, addition, restoration, and movement of structures and change of occupancy shall not be mandatory for historic buildings where such buildings are judged by the building official to not constitute a distinct life safety hazard. As such, the building official can allow the continued use of existing stairs, openings, and other features in historic buildings where compliance with the code would be damaging to historic features. In the case of the Gateway Arch, the authority having jurisdiction ultimately has the authority and discretion to utilize these code provisions regarding historic character to minimize impact on the structure.

The Life Safety Code (NFPA 101) Section 4.6.2 allows the authority having jurisdiction similar latitude as the IBC in the application of Life Safety Code requirements to existing historic structures. In doing so, the intent of

the current Life Safety Code is that requirements may be achieved through alternate equivalent methods resulting in similar levels of life safety.

The Secretary of the Interior's Standards for the Treatment of Historic Properties states that when Preservation is the appropriate treatment recommendation, while energy efficiency, accessibility and health/safety considerations are important they are:

... not usually part of the overall process of preserving character defining features; rather, such work is assessed for its potential negative impact on the building's historic character. For this reason, particular care must be taken not to obscure, alter or damage character-defining features in the process of preservation work.¹²⁵

Occupancy Type

For purposes of the scope of this study, the evaluation of occupancy type and related code information has been limited to general information about the structure.

Based on the IBC, the visitor center and Museum of Westward Expansion located at the base of the Arch contain a mix of occupancy types consisting primarily of A-2 and A-3 Assembly uses, with secondary uses consisting of B, Business; M, Mercantile; and S-1, Storage. The Arch, with its internal trams and stairs, offers access to the observation level at the top. This small indoor assembly area, which is used to view the surrounding river and cityscape adjacent to the monument, is most closely defined under the current building code as an A-5 Assembly use that includes such properties as amusement park

structures and structures designed to accommodate viewing of outdoor activities.

Construction Type

The visitor center, including the tram load zones, is an earth covered, windowless concrete structure consisting of what is most likely defined by the current IBC as either Type IA or Type IB non-combustible construction that is fully sprinklered for fire suppression. The Arch itself is not a typical building structure and is a combination of protected non-combustible concrete and unprotected steel that is best defined as Type II-B construction. The Arch is not sprinklered for fire suppression.

It should be noted that under the IBC, building structures are not allowed to be a combination of construction types. They must be designated to meet one specific set of construction requirements. The mix of construction types that makes up the Arch monument may be considered as acceptable if the Arch itself is viewed under the current rooftop structure requirements for towers, spires, domes, and similar rooftop features.

Building Height, Floor Area, and Occupant Load Requirements

The main portion of the structure is an underground building and, as such, has no real height as typically defined by the IBC. By defining the Arch itself as an A-5 occupancy, the number of stories is not limited. If the Arch is defined as a rooftop structure, the height and number of stories is also not regulated.

The actual floor area of the observation level located at the top of the Arch (at approximately 620 feet above grade) is approximately 440 square feet. The occupant load, calculated at one person per 5 square feet, is approximately eighty-eight people. It is important to note that the park staff does not

¹²⁵ Anne E. Grimmer and Kay D. Weeks, *The Secretary of the Interior's Standards for Rehabilitation & Illustrated Guidelines for Rehabilitating Historic Buildings* (Washington, D.C.: National Park Service, 1991), xii.

allow more than the number of people that the two trams will hold, plus two National Park Service employees, to access the observation level; this limits the occupancy at the observation level to approximately eighty-two people at any given time.

Building Code and Life Safety Issues

The following preliminary list of building code and life safety deficiencies has been identified. (As previously explained, the authority having jurisdiction may wish to defer or waive several of the following deficiencies in order to minimize impact on historic fabric if, in the authority's opinion, no distinct life safety hazard exists, or if the hazard has been mitigated by some other equivalent safety measure.)

1. Various existing stair treads, risers, and stair widths within the Arch legs do not fully conform to current requirements.
 - a. Most of the straight run stairs within the Arch legs have treads that are approximately 10-9/16 inches deep versus 11 inches as currently required by code. (The riser dimensions typically conform to current 7 inch maximum requirements.)
 - b. The width of the straight run stairs is approximately 2 inches too narrow (34 inches versus 36 inches required).
2. Not all of the existing handrails used at stairs and ramps leading to the main building entrances and within the building, as well as within the Arch legs, comply with current requirements.
 - a. Handrails are missing on exterior ramps leading to the main entrances.
 - b. The cast concrete handrails at the main entry do not provide required grasp dimensions or proper extensions.
 - c. Handrail extensions are missing on stairs within the Arch legs.
3. Many of the existing guardrails used at stairs and stair landings throughout the

Arch legs do not comply with current rail member spacing requirements. (This assumes that these guardrails should meet the requirements for public areas, as they would be key safety features for the public to use in the event of an emergency requiring the public to return to the base of the Arch by using the stairs.)

4. The interior and exterior ramps and landings connecting the main entrances to the tram load zones do not comply with current requirements.
 - a. With the exception of the exterior ramp leading to the main entrance (where the slope is approximately 12.6 percent), ramp slopes are generally within current requirements. However, all ramps have a rise between landings that exceeds the 30 inch maximum allowed by code.
5. Although not a specific building code requirement, a better system of smoke control within the Arch structure itself and the related tram load zones would greatly improve the safety of visitors/employees using the trams and observation level located at the top of the Arch.

Accessibility Review

The focus of this accessibility review is on the same areas described in the previous Code Compliance Review section of this report. The Architectural Barriers Act Accessibility Standards (ABAAS), as well as the ICC/ANSI A117.1 IBC requirements, have been used in the review.

National Park Service *Management Policies 2001* states in Chapter 5, Cultural Resource Management:

The National Park Service will provide persons with disabilities the highest feasible level of physical access to historic properties that is reasonable, consistent with the preservation of each property's significant historical features. However, if it is determined that modification of particular features would impair a property's integrity and character in terms of the Advisory Council's regulations at 36 CFR 800.9, such modifications will not be made.¹²⁶

In accordance with this policy, the Arch should, at a minimum, be made accessible from the exterior sidewalk areas at the main public building entrances and extending to the Arch tram load zones.

Deficiencies

The following deficiencies related to accessibility were noted:

1. Although accessible parking is beyond the focus of this study, it should be noted that it is currently located a considerable distance from the Arch entrances.
2. The exterior route connecting the immediate sidewalk area near the Arch base outside of the building and leading to the main building entrances is not accessible. The configuration of the

handrails and the length of the ramp before a landing is encountered do not comply with accessibility guidelines. The existing ramps are too steep to comply with accessibility guidelines.

3. The interior stairs, ramps, handrails, and landings do not provide an interior accessible route between the main building entrances and the tram load zones, and the observation level located at the top of the Arch. The ramp leading down to the visitor center is too steep and does not have proper spacing of landings. The handrail detail that is integral to the concrete guard wall does not comply with accessibility guidelines with regard to grasp ability. The ramps leading from the visitor center to the tram load zones do not meet accessibility guidelines with regard to proper spacing of landings nor does the ramp have hand rails. The route down to the queuing lines, which must be negotiated by stairs only, is not accessible to persons in wheelchairs. Finally, access to the tram capsules does not meet accessibility guidelines due to the height of the tram threshold.
4. The existing toilet room located at the top of the Arch is not accessible. The square footage devoted to the emergency toilet room is extremely tight and does not have the proper grab bars.
5. The interior stairs located in the Arch legs are not fully accessible.
6. Accessible signage is not adequately displayed.

¹²⁶ National Park Service, *Management Policies 2001* (Washington, D.C.: U.S. Department of the Interior, National Park Service, 2001), section 5.3.2, 53.

Life Safety Review

Exit and Emergency Lighting

The exit lighting fixtures have red letters and are illuminated with incandescent lamps. The exit lighting fixtures are located in the lower tram load zone, upper tram load zone, at the top of the Arch in the visitor observation area, and in non-public service corridors.

The battery powered emergency lighting fixtures have integral batteries and are the car headlight type. The emergency lights are located in the lower tram load zone, upper tram load zone, and at the top of the Arch in the visitor observation platform. The normal use lighting fixtures are also connected to the emergency generator distribution system. The emergency lighting fixture type, placement, and quantity meet life safety codes.

Fire/Smoke Detection and Alarm

The fire alarm system was updated in 2007. The new fire alarm system is a Simplex Voice Evacuation system. The main fire alarm control panel is located in the maintenance office. Manual pull stations are located in the upper tram load zone, at the top of the Arch in the visitor platform area, and back of house in corridors and mechanical room. Combination fire alarm speaker strobes are located in the upper tram load zone, lower tram load zone, top of the Arch in the visitor platform area, and back of house in corridors and mechanical rooms.

Smoke Evacuation System

The existing HVAC system incorporates a control sequence for smoke evacuation from the Arch in the event smoke is detected. At the top of the Arch just outside the observation level is a series of dampers and relief air louvers for pressure control and smoke evacuation. Under normal operation, some dampers are left open to relieve pressure from

the Arch legs, to allow for ventilation air to be introduced into the system. Smoke detectors are installed at the observation level, below the observation level in the tram electrical equipment space, and at the Arch leg air handlers. According to JNEM maintenance staff, in the event smoke is detected the following sequence is carried out by the controls system and components:

- The north and south Arch leg air handler supply fans go to 100 percent.
- The north and south Arch leg air handler return fans shut down.
- The outdoor air dampers at the Arch leg air handlers open to 100 percent.
- The dampers at the top of the arch open to 100 percent to relieve all of the air flowing into the Arch via the Arch leg air handlers out of the louvers on the Arch exterior.

This control and operation sequence then forces smoke out of the Arch legs and observation level by pressurization.

The dampers and control components could not be closely inspected during the investigation. Upon visual inspection through the plexiglass windows from the observation level, it appears the dampers and actuators are in fair condition. This assessment is due mostly to the age of the system. No operational deficiencies were observed or reported by park maintenance staff.

Hazardous Materials

During the initial stages of this study, JNEM personnel were interviewed to obtain information about past investigations and to learn whether hazardous materials had been discovered or are present within the Arch HSR study areas. Park maintenance staff confirmed that the presence of lead containing paint had been verified several years ago at the interior of the Arch legs. Park staff indicated that the finish paint coats covering all of the interior

surfaces including the inner skin carbon steel plates, beams, and stiffeners as well as stair and tram structural components contain lead.

The tram cabins or capsules have been through several maintenance cycles during which the capsule interiors and exteriors were stripped of all paint, cracks were touched up and welded, and finally the interior and exterior steel surfaces were repainted. The interiors of the capsules are regularly repainted to cover graffiti and vandalism, and the paint layers become quite thick, necessitating periodic removal of all paint prior to recoating. As part of the Historic Structure Report study, paint samples were taken from other painted surfaces including handrails at tram load zones, the interior painted steel surfaces of the observation level, and upper tram load zones. These were analyzed for paint color and chronology as discussed in the Finishes Analysis section of this report; however, analysis for hazardous content was not part of this scope of services.

No reports have been obtained that indicate the presence of any asbestos containing materials or other hazardous materials, except for pipe wrapping as mentioned in the mechanical section of the HSR. The aforementioned occurrence is noted and tagged as *possibly* containing asbestos

PART 2 – TREATMENT AND USE



Jefferson National Expansion Memorial Gateway Arch. Source: HABS photograph by Jack Boucher, 1986.

HISTORIC PRESERVATION OBJECTIVES

The U.S. National Park Service has developed definitions for the four major treatments that may be applied to historic structures: preservation, rehabilitation, restoration, and reconstruction. The four definitions are provided below for reference:

Preservation is defined as the act or process of applying measures necessary to sustain the existing form, integrity, and materials of an historic property. Work, including preliminary measures to protect and stabilize the property, generally focuses upon the ongoing maintenance and repair of historic materials and features rather than extensive replacement and new construction. New exterior additions are not within the scope of this treatment; however, the limited and sensitive upgrading of mechanical, electrical, and plumbing systems and other code-required work to make properties functional is appropriate within a preservation project.

Rehabilitation is defined as the act or process of making possible a compatible use for a property through repair, alterations, and additions while preserving those portions or features which convey its historical, cultural, or architectural values.

Restoration is defined as the act or process of accurately depicting the form, features, and character of a property as it appeared at a particular period of time by means of the removal of features from other periods in its history and reconstruction of missing features from the restoration period. The limited and sensitive upgrading of mechanical, electrical, and plumbing systems and other code-required work to make properties functional is appropriate within a restoration project.

Reconstruction is defined as the act or process of depicting, by means of new construction, the form, features, and detailing of a non-surviving site, landscape, building, structure, or object for the purpose

of replicating its appearance at a specific period of time and in its historic location.¹²⁷

When the property's distinctive materials, features, and spaces are essentially intact and thus convey the historic significance without extensive repair or replacement; when depiction at a particular period of time is not appropriate; and when a continuing or new use does not require additions or extensive alterations, **Preservation** is considered an appropriate treatment.

Preservation is defined as the act or process of applying measures necessary to sustain the existing form, integrity, and materials of an historic property. Work, including preliminary measures to protect and stabilize the property, generally focuses upon the ongoing maintenance and repair of historic materials and features rather than extensive replacement and new construction. New exterior additions are not within the scope of this treatment; however, the limited and sensitive upgrading of mechanical, electrical, and plumbing systems and other code-required work to make properties functional is appropriate within a preservation project.

1. A property will be used as it was historically, or be given a new use that maximizes the retention of distinctive materials, features, spaces, and spatial relationships. Where a treatment and use have not been identified, a property will be protected and, if necessary, stabilized until additional work may be undertaken.
2. The historic character of a property will be retained and preserved. The replacement of intact or repairable historic materials or alteration of features, spaces, and spatial relationships that characterize a property will be avoided.

¹²⁷ *The Secretary of the Interior's Standards for the Treatment of Historic Properties.*

3. Each property will be recognized as a physical record of its time, place, and use. Work needed to stabilize, consolidate, and conserve existing historic materials and features will be physically and visually compatible, identifiable upon close inspection, and properly documented for future research.
4. Changes to a property that have acquired historic significance in their own right will be retained and preserved.
5. Distinctive materials, features, finishes, and construction techniques or examples of craftsmanship that characterize a property will be preserved.
6. The existing condition of historic features will be evaluated to determine the appropriate level of intervention needed. Where the severity of deterioration requires repair or limited replacement of a distinctive feature, the new material will match the old in composition, design, color, and texture.
7. Chemical or physical treatments, if appropriate, will be undertaken using the gentlest means possible. Treatments that cause damage to historic materials will not be used.
8. Archeological resources will be protected and preserved in place. If such resources must be disturbed, mitigation measures will be undertaken.¹²⁸

REQUIREMENTS FOR WORK

Guidelines and Standards for Treatment

Guidelines and requirements for treatment have been defined based on the preservation objectives outlined above for the Gateway Arch. All treatment guidelines and recommendations were developed in accordance with the Secretary of Interior's Standards for Preservation.

The approach presented combines recommendations for preservation and conservation of original materials and features, together with repairs and improvements to meet building system and code compliance requirements. In addition, continued ongoing maintenance of the systems and materials of the Arch is required as part of overall preservation. As the recommended work will likely be performed as part of ongoing preservation efforts rather than as a single comprehensive project, prioritization or phasing of specific recommendations has been addressed for purposes of this study.

1. Protection of Primary Structural Elements. Studies and recommended investigation and repair, as related to protection of the primary Arch structure from deterioration, should be undertaken.
2. Life Safety and Functionality Upgrades. Designs for appropriate life safety and functionality upgrades to the Arch should be studied and developed, with due consideration of the effect of any changes on the historic character-defining features of the Arch.
3. Restoration. Where altered, original interior finish materials and surfaces should be restored to a condition closer to the original design intent, including materials, textures, and color.

¹²⁸ Ibid.

4. Cyclical Inspection and Maintenance. In addition to the specific repairs recommended, cyclical maintenance tasks such as inspection, painting of exposed steel elements, cleaning, repair, and/or replacement of finishes in the primary public areas of the arch, and other ongoing maintenance tasks should be continually implemented to avoid damage to the historic building fabric and to reduce the need for large-scale repair projects in future.

All work performed on the Gateway Arch should be documented through notes, photographs, and measured drawings and/or sketches, and by as-built annotations to construction documents at project completion. These records should be permanently archived as a record of the work, for future reference, and to provide information for future maintenance of the Arch. In addition, these records will allow future observers to identify which materials and system components are original and to understand the chronology of repairs and other changes that have occurred to the structure over time. It is recommended that these records be archived at JNEM and also included in another collection, such as the NPS Denver Technical Information Center, for reference.

RECOMMENDATIONS

Specific recommendations are presented in the following sections.

Major repair work as well as ongoing maintenance should be documented. A permanent record of the actions taken and the methods, materials (including identification of any proprietary products), and equipment used should be maintained at the park. The documentation should cover architectural and structural changes as well as work related to the mechanical and electrical systems.

Exterior

The Gateway Arch is a unique monument in typology and material. Due to its distinctive character, a more scientifically informed understanding of its behavior needs to be reached with comprehensive inspections and observations/analysis as part of the development of recommendations for its preservation. Findings of the physical investigation recommended below could become a supplement to the HSR. Including further archival research, visual inspection, laboratory testing, long term monitoring, and analyses.

Close-up access to the various faces of the Arch will be necessary. Close-up access would provide opportunities for visual and microscopic inspections, inspection of welds, non-destructive sample removal of stains and discolorations, sample collection for laboratory analysis, measurements for surface tolerances and deformations, and treatment mock ups.

Access to the upper reaches of the Arch requires ongoing consideration. The designers of the Arch provided no means of exterior access for future maintenance. Types of close-up access previously considered include cranes, scaffolding, industrial-rope access (rappelling), and helicopters. Each of these techniques has shortcomings and limitations. Cranes will only rise to approximately 300 feet. Scaffolding would be extremely expensive. Rappelling is risky, and access would be limited to experienced climbers. Helicopters would provide a limited degree of closeness that would ultimately restrict collection of information.

One initial solution would be to install small stainless steel anchor clips to allow limited access to some faces containing discoloration and distress (Figures 256 and 257). The use of

stainless steel anchors will greatly facilitate scheduled exterior inspections.

Improper introduction of these anchors can create new sites with lower corrosion resistance and thus jeopardize the exterior's longevity. However, using matching stainless steel, good welding practice, and thorough post-welding cleaning will minimize potential for corrosion at these points.



Figure 256. Stainless steel anchor proposed as option of permanent attachment to be installed on the exterior skin of the structure to assist in exterior inspection of structure.



Figure 257. Stainless steel anchor proposed as option of permanent attachment to be installed on the exterior skin of the structure to assist in exterior inspection of structure.

Further investigation from the interior of the Arch should also be conducted to resolve unknown and potential deterioration conditions, for example, the condition of materials in the interstitial space between the inner and outer skins above and below the 300 foot level. This investigation would require openings to examine concealed conditions and materials.

Investigative Procedures on the Interior

The following procedures are recommended on the Arch interior based on location within the structure.

- The interior Arch surfaces are coated with lead-containing paint systems that must be abated prior to undertaking actions noted below. Selection of drill points/openings will need to be made ahead to allow NPS staff time to remove/abate the existing coatings as required.
- Stations or Segments 1 to 10 (top of Arch near louvers): Drill or cut through the interior skin in selected areas and inspect the interstitial space with a fiber optic borescope, or create small viewing ports. Access the interstitial space through louvers and inspect the louver openings. Measure the corrosion of the interior surfaces, if possible. Examine cut samples of steel for corrosion.
- Segments 20 to 30 (above concrete fill): Drill or cut through the interior skin in selected areas and inspect the interstitial space with an endoscopic device or create small viewing ports. Measure the corrosion of the interior, if possible. Examine cut samples of steel for corrosion.
- Segments 43/44/45 (top of concrete fill): Drill through the interior skin in selected areas and inspect the interstitial space with

an endoscopic device or create small viewing ports. Remove samples of concrete for moisture and chloride testing. Measure the corrosion potential and rate of the interior concrete.

- Segments 48 to 55+ (below concrete pour): Examine, clean, characterize, and photograph the corrosion of through-skin bolts. Select bolts for removal and further examination. Inspect interior space and interface between concrete and skin for sources of moisture. Sound interior surface (especially near bottom of segments) for unconsolidated concrete voids. Use impact echo method or similar nondestructive testing technique to verify the presence of voids.
- Base Segment (interface between Arch and foundation concrete): Inspect the corrosion near base, clean, characterize, and photograph. Drill through the inner shell and remove concrete samples for laboratory testing for moisture and chlorides. Measure the corrosion potential and rate of the interior concrete. Inspect the interface between the concrete and interior skin. Sound interior surface (especially near bottom of segments) for unconsolidated concrete voids. Use impact echo method to verify the presence of voids.

Laboratory Analysis

Removal of stainless steel samples from the Arch will require planning and discussion, as it is desirable from a preservation perspective both to limit removals as much as possible and to obtain information important to future conservation of the Arch. For initial testing purposes, it may be most practical to use a segment (including the weld) removed from the sample of material retained from original construction at the JNEM archives. During the detailed inspection of the actual structure,

small corroded stainless steel/weld samples cut from the skin would be useful. Samples would be removed from inconspicuous locations but would need to exhibit the phenomena that are being investigated. The samples would be cut as a rectangle or circle, would need to be removed from the exterior, would have to penetrate the skin, and would be repaired by welding on a replacement piece.¹²⁹

Samples of carbon steel cut from the interior would be removed, examined, and cleaned to quantify corrosion. Concrete samples would be tested for moisture content, humidity and presence of chlorides.

Assuming that the archive samples of the welded pieces of stainless steel and mild steel are determined to be representative of the structure, one to two inch slices of the archive samples will be tested in the following manner. The slices would be exposed to accelerated corrosion testing and then examined using metallographic techniques. Corrosion of the steel, weld material, and the heat-affected zone (HAZ) of the base metal will be examined.

Chemical analysis of the stainless steel would be performed. Samples of various stains or discolorations would be obtained for analysis by X-ray diffraction, scanning electron microscope, infrared, or atomic absorption.

Long Term Monitoring Program

The following monitoring program is recommended that can be installed in tandem with the interior investigative procedures described above.

- Stations or Segments 1 to 10 (top of Arch near louvers): Install long term air temperature and moisture sensors within the interstitial space and inside monument at each leg. Install surface temperature sensors at each of the three faces of the carbon steel at each leg.
- Segments 20 to 30 (above concrete fill): Install long term air temperature and moisture sensors within the interstitial space and inside monument at each leg. Install surface temperature sensors at each of the three faces of the carbon steel at each leg.
- Segments 43/44/45 (top of concrete fill): Install long term air temperature and moisture sensors within the interstitial space and inside monument at each leg. Install surface temperature sensors at each of the three faces of the carbon steel at each leg.
- Segments 48 to 55+ (below concrete pour): Install surface temperature sensors at each of the three faces of the carbon steel at each leg. Install moisture sensors within the monument at each leg.

Development of an “Institutional Memory” Database

Many of the phenomena that were observed during this investigation were observed by others in the past. This is evident from discussions in construction correspondence in archives, past monitoring numbers and notes in place on the interior carbon steel wall, and our discussions with JNEM personnel. To date these observations have not been recorded or

¹²⁹ Cutting and patching stainless steel samples from the exterior of the Arch is unfortunately the only way presently to determine definitively the composition of the skin. The cutting and patching process does present challenges from a logistical and access standpoint and would require Section 106 review. Sampling would need to be carefully evaluated as to its appropriateness. Any proposed work should utilize the gentlest methodology.

coordinated in a formal fashion. A more focused, and directed monitoring program should be established over the long term. The Arch should be periodically inspected and photographed for corrosion and stain progression.

Preparation for Cleaning

As discussed above and in the previous *Gateway Arch Corrosion Investigation, Part I*, dated May 2006, corrosion of stainless steel is promoted by soil on the surface that collects contaminants, promoting formation of concentrated electrolyte and inhibiting natural repair of the chromium oxide film. It is important, therefore, to ensure that the exterior stainless steel surface is clean and uncontaminated. This enables the inherent corrosion resistance conferred by the additions of chromium, nickel, etc., to the stainless steel alloy to be fully realized. A primary treatment to prevent corrosion and maintain stainless steel is to keep the surface clean.

It is recommended that the Arch be cleaned within the next ten years and that it be cleaned on approximately a fifty year cycle thereafter. The initial cleaning will be expensive due primarily to access requirements, but a reusable means of access should be designed as part of the cleaning procedure. This access will then provide for close up inspection of the Arch exterior at least during each cleaning and possibly more frequently if required. Refer to discussion above and to Figures 250 and 251 for clip anchors which may aid in survey and future cleaning procedures.

Note that some variability of the Arch surface, such as the dimpling and oil canning of the surface related to the locations of internal supports, was apparent during initial construction and was accepted as visual evidence of the means of construction of the structure. It would not be appropriate to

attempt to alter or treat the surface to eliminate or reduce these visual effects.

Maintenance Cleaning and Graffiti Control

Frequent mild cleaning of the stainless steel using warm water and mild, pH neutral detergents can be used for routine maintenance to remove aerosol chlorides, atmospheric pollutants and oils from fingerprints, etc. Cleaning trials and mock-ups should be conducted to identify the most appropriate materials and techniques for routine cleaning.

During construction minor abrasions were cleaned using “a fine grit impregnated cloth.”¹³⁰ Weld halos from shop and field welding were cleaned using an electrolytic process, and grease and general dirt accumulations were originally removed using a solution of Oakite No. 33.¹³¹ According to the current Material Safety Data Sheet, Oakite No. 33 is a phosphoric acid based cleaner. It is unknown if this is the same formulation as was available in the 1960s.

The incised graffiti presents a unique challenge to the maintenance of the Arch. While abrasive cleaning techniques such as non-woven pads, abrasive wheels, etc., have reportedly been used in the past to remove isolated areas of incised graffiti, these techniques remove portions of the historic fabric of the Gateway Arch. Evidence of these techniques is visible on the south leg on the

¹³⁰ Letter from K. J. Kolkmeier of Pittsburgh-Des Moines Steel Company to B.A. Prichard of MacDonald Construction Company, dated December 5, 1963. The cleaning methods were approved for use by NPS, letter from H. Raymond Gregg of JNEM to R. E. MacDonald, dated February 20, 1964. The specific cleaning procedures were developed by PDM after preliminary tests of various protective coverings for the stainless steel Arch surface were found to be unsatisfactory.

¹³¹ Ibid.

west side of the Arch. Abrasive cleaning techniques are not recommended as they will remove historic fabric and will not promote the long term conservation of the Arch. Repeated grinding and polishing could result in an unacceptable loss of section. A light polishing may be beneficial with the intent of improving the appearance of the surface without significant section loss. The rate of new incised graffiti is unknown and should be monitored.

The *Gateway Arch Corrosion Investigation, Part I* report also suggested a wax treatment or welding a very thin stainless steel covering that could become sacrificial as techniques to minimize the visual appearance of the incised graffiti. Further, a film forming clear coat could also be considered. The coatings, either wax or film forming, may make the incised graffiti more visible. The thin stainless steel covering may lead to unacceptable oil canning between the covering and the original historic fabric of the Gateway Arch. Samples, mock-ups and trial repairs are recommended as part of a future study on the investigation and mitigation of the incised graffiti.

Deicing salts should not be used on the exterior areas adjacent to the Arch. Salts could accelerate corrosion of the exterior skin, and salt-laden water could also accelerate deterioration of interior steel plates and possible deterioration of reinforcing bars at the base of the Arch.

Interior

North and South Tram Load Zones

The following recommendations include the north and south tram load zones at the base of the Arch, including both the upper and lower spaces, stairs, and connecting areas.

Both the north and south tram load zones were designed originally as voluminous interconnected spaces characterized and defined by simple modern materials—architectural concrete walls and balcony railings, terrazzo floors and stair surfaces, and exposed waffle slab concrete structure at the ceilings. These spaces gave a visitor a definite sense of progression as one descends down to the base and foundation of the Arch and is then loaded onto small capsules for a ride to the very top of the Arch. The sense of volume and spatial definition imparted by these spaces has been somewhat compromised by the recent modifications described above. Also, as the interpretive needs for the park have evolved, additional materials have been installed covering the original exposed concrete wall surfaces, balcony details, and exposed structure.

Future treatment recommendations for these spaces will likely be affected by directives provided in the General Management Plan, which was finalized and issued in October 2009, and the resulting exhibit and interpretive plans. The preferred alternative identified in the GMP calls for redesigning exhibits and providing more interactive presentations for visitors to the Museum of Westward Expansion and other areas, which would likely change or redefine how these transitional spaces are interpreted. As displays and interior finishes are replaced or altered in the future, the underlying original interior finishes and structure should be considered, and new work should be installed in a manner that protects original materials from damage. As

interpretive improvements are made at other areas, it may be possible to expose more of the original wall surfaces and ceilings to give the feeling and character of the original modernist space.

In implementing repairs, priority should be given to the areas in the tram load zones that are experiencing water infiltration at the expansion joints and the wall to ceiling interface in order to address these problems.

Specific treatment recommendations for the materials and surfaces found in the tram load zones are as follows:

Walls

- The architectural concrete, which is a significant design feature, acts as both the structural material and finish material. As much of the original concrete surface as possible should be retained and exposed in the tram load zones, as this expression was important original feature.
- Additional research and testing is recommended to determine what sealer was used and the most effective and gentlest methods of cleaning the concrete surfaces. This information should be used in developing a standard for future maintenance cleaning procedures.
- Determine if this sealer will discolor and if it is possible to remove in future if needed.
- For cyclical maintenance cleaning, use mild detergents and water to remove dirt and soil. Generally follow recommendations found in NPS Preservation Brief 15, *Preservation of Historic Concrete*.¹³² Occasionally it may be necessary to remove more severe stains or graffiti, which may require stronger cleaning methods such as chemical

¹³² Paul Gaudette and Deborah Slaton, *Preservation Brief 15: Preservation of Historic Concrete* (Washington, D.C.: U.S. Government Printing Office, 2007).

cleaning products or poultices. Abrasive cleaning techniques that may abrade or remove form markings in the architectural concrete surfaces, or discolor original concrete coloring/pigments, should be avoided.

- For patching of spalls, holes, and deteriorated concrete areas, samples of architectural concrete should be examined in the laboratory to determine an exact mix design, including selection of matching aggregate, paste, and color and texture of repair mixes. Trial samples should be performed to help develop the mix design and finishing technique, and to prepare a standard specification for all future architectural concrete repairs. The repairs should be finished to match the texture of the adjacent original concrete.

Floors

- Areas of terrazzo exhibiting distress should be removed and replaced with new cement terrazzo formulated to match the existing terrazzo. If a large area of terrazzo is deteriorated, it should be replaced in sections corresponding to the existing grid formed by the divider strips. A partial repair should include removing the distressed area and saw-cutting to form square edges. The exposed surface should be roughened using abrasive blasting methods and repaired using a terrazzo mix with a latex bonding agent. Large cracks in the terrazzo should be treated in the same manner as described above. Following repair of the terrazzo, the surface should be cleaned by removing the surface sealers, rinsed with a non-ionic detergent, and resealed with a clear penetrating sealer. Detailed recommended treatment of the terrazzo flooring in the

tram load zones has been addressed in a previous study commissioned by NPS.¹³³

- Future flooring finishes work should take into consideration the original underlying materials, and new flooring systems should be installed in a manner that protects original materials from damage. For example, use of removable adhesives would be preferable to mechanical attachment of floor coverings, if installed in the future.

Ceilings

- For the exposed painted concrete structure, the painted surface should be cleaned with mild detergents and water as needed. As repairs are required, concrete patching techniques should be implemented as described above for the concrete wall systems. Paint coatings should be touched up as needed. The ceilings should be repainted as required utilizing appropriate paint systems for concrete surfaces.
- For the suspended acoustical tile ceiling systems, maintenance should include cleaning and replacement of damaged and deteriorated units with new units that match the existing system components.

Hollow Metal Door Systems

- Painted hollow metal doors and frames should be maintained with routine cleaning of surfaces and touch up painting as needed.
- Damaged/rusted door frames or assemblies may require cutting and patching in of new metal, priming and painting.

¹³³ NPS Jefferson National Expansion Memorial Gateway Arch; Lobby, Restroom and Terrazzo Repair/Restoration –Final Report Title I, March 2008.

Tram Doors/Entrance Systems and Tram Capsule Interiors

- Tram loading doors and metal surrounds are original to the transportation system and are important character defining features of the space. The doors and surrounds have recently received extensive work including stripping of all paint layers to bare metal and installation of new primer and paint finish. For more discussion regarding the finishes of the tram capsules refer to Appendix B for more detail. Touch up of these areas is recommended on an as-needed basis with the paint systems matching the existing color, texture, and finish.
- The tram cabin interiors are also a very important part of the Arch experience and retain a great deal of integrity. The cabin interior surfaces receive daily as well as annual maintenance. Due to the constant use and visitor graffiti, the interiors receive touch-up painting and are totally repainted once or twice annually. Over the life of the trams, the steel interior cabin surfaces have been stripped of all paint layers several times to remove extensive paint build up and to facilitate repair and welding of cracks in the steel. Touch up of these interior surfaces is recommended on an as-needed basis with the paint systems matching the original color, texture, and finish. Stripping and repainting as is currently practiced is also appropriate when needed to reduce paint build-up and to remove graffiti if extensive. The original contoured plastic seats should be retained and repaired as needed. If replacement is required due to damage or deterioration, replacement with seats that match in color, material, and design is recommended. If any of the seats are removed for replacement, they should be placed in park collections.

Observation Level

At the top of the Arch, the observation level retains a fair amount of integrity in terms of the spatial character and configuration of the spaces. The inner walls of the observation level are essentially the inner carbon steel layer of the triangular Arch structural sections, except for the areas around the viewing windows where steel plates form a small continuous step under the windows, and the steel soffit panels and light valances at each wall. The finishes of the sloping outer walls and viewing niches and floors have undergone many cycles of finish material removal and installation. The original Saarinen construction drawings call for the interior steel wall surfaces of the observation level to have a vinyl plastic finish.¹³⁴ The original floor materials at the observation level were specified to be an abrasive steel plate. The project specifications (Section 7-4.5) call for this plate to contain aluminum oxide particles cast into the steel plate as an integral finish. This palette of finishes would impart a very sleek and clean, crisp aesthetic to the space and to the finely detailed exposed steel elements. Due to the large amount of visitation and vandalism, the vinyl wall finish has either been removed or has been covered with other materials. For many years the park has been covering the walls at the viewing niches and outer walls the tram load zone with a durable grade of wall carpet, and covering the observation level floor with sheet goods carpet. These carpet coverings are treated as a sacrificial surface treatment; that is, once the carpet material is worn and deteriorated it is removed and replaced. This approach has served the park well over the past many years for several reasons. The carpet surfaces act as a comfortable surface on which visitors can lean as they look out the windows to the landscape below. The carpet also acts as an

¹³⁴ Original specification, Section 7-2.8.c.

acoustical treatment to deaden the sometimes loud sound levels that occur when there are many visitors at this level. At the time of this study, JNEM maintenance staff mentioned that the existing carpet materials were scheduled for replacement within the coming year. Although not the historic finish material, the carpet is an easily reversible added material that serves to protect original materials from excessive wear.

The following are some potential treatment options for the observation level and tram load zones:

Wall and Floor Surfaces

- The current policy of installing and removing carpet on the walls and floors is necessary due to the heavy wear received in this area, and has not damaged the underlying steel wall panels to any extent visible during this study. Although transporting materials up to this level is difficult, it continues to be a routine procedure.
- The current treatment approach described above can be continued. However, due to the chance of fire and subsequent development of smoke on the observation level, it would be advisable to utilize a carpet material that complies with the smoke and flame spread testing of ASTM E 84, *Standard Test Method for Surface Burning Characteristics of Building Materials*, to mitigate the potential for the carpet material to produce noxious fumes. Also, the carpet material should comply with ASTM E 648, *Standard Test Method for Critical Radiant Flux of Floor-Covering Systems Using a Radiant Heat Energy Source*, (class II). Compliance with this testing procedure will mitigate the potential for combustion of the carpet material from radiant heat transfer due to fire in the equipment space below the observation level. The carpet specified for the observation level currently meets the aforementioned ASTM references, however, any deviation from the use of the currently specified carpet material should continue to comply with the ASTM references.
- It is desirable that the new carpet have a high content of recycled material, if possible.
- Also a wall carpet color and texture should be selected that do not compete with or distract from the character of the space. The color should be neutral and the carpet should be able to tolerate the abuse it receives from hundreds of visitors. Ease of installation and removal should also be considered, as well as the use of adhesives that are acceptable in tight and enclosed spaces with little available ventilation.
- If continued research can indicate the nature and composition of the original vinyl plastic finish, consideration should be given to restoring at least a portion of the wall surface to this original material. Appropriate areas for finish restoration could include the outside wall area at the observation level tram load zone opposite the loading doors, and the vertical steel V-shaped panels separating the viewing windows. Wall carpet could continue to be used on the areas where visitors lean or rest.
- Further research and investigation should be conducted to confirm the nature and composition of the original abrasive steel finish at the steel flooring and its underlying condition. Further research should be conducted when the carpet is removed for maintenance and/or replacement. The stair treads at the observation level tram loading stairs are thought to be the original abrasive finish. This finish should be maintained and preserved.
- The observation level floors, which originally had an abrasive steel finish,

have been carpeted for reasons of acoustics and maintenance. This treatment approach is considered appropriate, although the same comments as made for wall carpet pertain to floor carpet—especially in terms of color, texture, flame and smoke spread ratings, and desirability of recycled content. Other flooring options include monolithic sheet goods such as non-slip rubber, which could impart a similar aesthetic as the original finish. This type of finish would not be as quiet but may be more durable than the carpet presently used.

Ceilings

- The exposed steel finish on the observation level ceilings should be maintained. Routine cleaning of painted steel surfaces and touch up painting should be performed as required.

Windows

- The stainless steel frame and 3/4 inch plate glass windows are important character defining features, and should be maintained.
- The windows receive routine maintenance by park staff, including cleaning, reglazing, gasket maintenance and replacement, and lubrication of hinges. These activities should be continued to keep windows in serviceable and operable condition.
- In-kind replacement should be performed only of severely deteriorated components. Replacement units should match the existing components in design, material, and color/composition.

Emergency Restroom

- The original steel finish should be maintained on the walls and vertical steel surface. Routine cleaning of painted steel surfaces and touch up painting should be performed as required.

- The steel plate floors are thought to be the original abrasive finish. This finish should be maintained and preserved.
- The exposed steel finish of the ceiling should be maintained. Routine cleaning of painted steel surfaces and touch up painting should be performed as required.

Control Booth

- For the steel surface of the walls and control rack, the original steel finish should be maintained. Routine cleaning of painted steel surfaces and touch up painting should be performed as required. The non-original wood grained vinyl folding door should be removed. If equipment security is of concern, a clear security covering should be installed over the equipment to prevent visitors from tampering with controls.
- The steel plate floors are thought to be the original abrasive finish, and as such this finish should be maintained and preserved.
- The exposed steel finish of the ceiling should be maintained. Routine cleaning of painted steel surfaces and touch up painting should be performed as required.

Interior Areas of Arch Legs

- Painted finishes should be maintained. Routine inspections of all elements should be performed to detect corrosion and/or any deterioration of the finish.
- State and federal procedures for lead based paint abatement have reportedly been followed by park staff and should be adhered to, should repairs be needed at select areas.

Maintenance Areas at Base of Arch Legs

Walls

- Cleaning and maintenance should consist of regular cleaning with detergents and water to remove dirt and grime. Concrete

walls are generally painted or unfinished non-architectural concrete.

- Spalls, holes, and areas of deteriorated concrete should be patched using a mix design that matches the original concrete in terms of aggregate, paste, color, and finish, as discussed above.
- Walls that are currently painted and will remain painted should be touched up or repainted as necessary.

Floors

- Maintenance should include regular cleaning with mild detergents and water to remove dirt and soil. The concrete floors are generally sealed or unfinished concrete.
- Spalls, holes, and areas of deteriorated concrete should be patched using a mix design that matches the original concrete in terms of paste, aggregate, color, and finish, as discussed above.
- Provide proper maintenance for sealed floors, including removing and replacing the sealer as necessary.

Structure

The following structural treatment and use recommendations for the continued preservation of the Gateway Arch were developed based on the information obtained from the structural condition assessment; review of previous of structural studies, original drawings and specifications; and interviews conducted with various representatives of the design and construction teams.

The following is recommended:

1. Conduct further research to locate previously completed structural studies that were unavailable during this investigation. These studies include the following:
 - Report by D.B. Steinman, Consulting Engineer, New York, *Jefferson Memorial Arch—Aerodynamic Studies*, December 31, 1948. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964, but was not available for review during this study.
 - Report by D.B. Steinman, Consulting Engineer, New York, *Jefferson Memorial Arch—Supplementary Aerodynamic Studies*, February 24, 1949. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964.
 - Report No. 314-001 by Fairchild Aircraft and Missiles Division, *Jefferson National Expansion Memorial Arch—Dynamic Analysis*, April 27, 1960. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964.
 - Report No. 314-002 by Fairchild Stratos Corporation, *Jefferson National Expansion Memorial Arch—Dynamic Analysis*, October 23, 1961. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964.
 - Undated report in German, entitled *Sicherheitsnachweis für seitliches Ausknicken des Bogens [Safety Certification against Sideways Buckling of the Arch]*, by Dr. Konrad Sattler, Technical University, Berlin. This document is referenced in the bibliography of the Bureau of Reclamation report of December 1964.
2. Monitor temperature and relative humidity within each leg of the Arch for comparison with exterior values.
3. Perform routine maintenance on mechanical equipment to ensure that issues with steam radiator piping and other mechanical systems do not contribute to excessive condensation within the legs of the Arch.
4. Perform routine evaluations of the field welds from the interior, in order to ensure no excessive deterioration is occurring because of excessive thermal stress concentrations.
5. Provide adequate drainage from horizontal surfaces, primarily at the location of the transition from the interior steel plates to the concrete foundation walls, in order to ensure that the potential for further corrosion is minimized.
6. Identify all fasteners with visible surface corrosion; mechanically remove the surface corrosion and coating; and evaluate for section loss of the fasteners or tie rods. If section loss is not apparent, clean and paint exposed fasteners and tie rods with a corrosion-inhibiting coating to prevent further corrosion. If significant section loss is exhibited, the fasteners/tie rods should be evaluated for adequate strength. Samples of the fasteners experiencing significant surface corrosion can be removed for laboratory testing (including metallurgical and tensile testing) to determine the capacity of the fasteners in the current state. If the required strength does not exist these fasteners should be removed and replaced.

7. Perform routine evaluations of the fasteners (bolts, field welds, shop welds, plug welds, etc.) from the interior for surface corrosion, deformation, and looseness. Compare with previous findings to evaluate the extent of new deterioration caused by moisture infiltration and movement of the structure.
8. Perform non-destructive testing of the fasteners (bolts, field welds, shop welds, plug welds, etc.) from the interior to identify any continued deterioration.
9. Identify visible surface corrosion on interior plates; mechanically remove the surface corrosion and coating; and evaluate for section loss. If section loss is not apparent, clean and paint exposed plates with a corrosion-inhibiting coating to prevent further corrosion. If significant section loss is experienced the plates should be evaluated for adequate strength. If the required strength does not exist, additional plates should be installed at the interior in order to obtain the required section modulus.
10. Sounding with a hammer was performed on the surface of the interior plate at the locations where corroding fastener heads were found. At these locations, delamination (an air gap) exists between the concrete and the interior carbon steel skin. It is recommended that sounding with a hammer of the interior plates should occur at all accessible locations of the interior plates, to check for and indicate locations of delaminations between the interior skin and concrete fill.

Mechanical, Electrical, and Plumbing

Heating, Ventilating, and Air Conditioning

The existing heating, ventilating, and air conditioning (HVAC) systems serving the Arch are generally in fair condition.

Continued maintenance on the north and south Arch leg air handlers should be sufficient for preservation of the current equipment. As there is currently a work order to replace the steam heating coils within the Arch leg air handlers, the existing chilled water cooling coils and associated condensate drain pans should also be thoroughly inspected for any signs of deterioration or leaking. Should any deficiencies in the chilled water coils be noted, they should be replaced with new coils.

Motor horsepower for the north and south Arch leg air handlers' supply and return fans should be recorded and posted at the respective air handler section and/or associated variable frequency drive.

The existing dual duct mixing boxes serving the Arch legs should be thoroughly inspected for leakage and functional deficiencies. The mixing box dampers and actuators should also be tested to ensure proper operation. The boxes should be re-sealed as required and actuators should be replaced as required. This work, along with continued maintenance to the existing mixing boxes, should be sufficient for preservation of the current system.

The current capacity and humidity control capabilities of the existing Arch legs HVAC systems are of concern. The Arch legs are not currently conditioned for any level of occupant comfort, as they are an unoccupied space. As mentioned earlier within this report, the methodology used for the design and sizing of the current systems is unclear due to lack of archival design information. Climatic condition information has been gathered for

the interior of the Arch legs in an effort to understand the condensation problems. An additional HVAC study is recommended to investigate further the climatic problems within the Arch legs and the feasibility of HVAC system replacement or expansion in order to control or resolve the current moisture issues.

Continued maintenance of the existing air handlers, condensing units, and mixing boxes serving the lower tram loading areas should be sufficient for preservation of these systems.

The existing chilled water piping system, as well as the existing steam and condensate piping systems, should be thoroughly inspected for leaks due to the age of the piping. The existing insulation on these piping systems should be inspected for possible asbestos containing insulation materials. Asbestos material inside the building is a health issue. An asbestos abatement investigation of the north and south mechanical rooms may be warranted in order to identify and remove possible asbestos from the existing insulation systems. This investigation should encompass all piping located in the north and south mechanical rooms and throughout the facility, not only that exclusively serving the Arch itself.

Consideration should be given to providing new preformed fiberglass pipe insulation for the existing chilled water, steam, and condensate piping systems. Minimum insulation thicknesses for piping should be as required by the International Energy Conservation Code. This insulation upgrade could be performed in conjunction with an asbestos abatement project.

Continued maintenance of the existing chilled and condenser water pumps and related equipment, and the existing cooling tower

should be sufficient for continued use of these systems.

The existing pneumatic temperature control system and components should be completely removed, and the existing control system should be upgraded to a full direct digital controls (DDC) system. Many options for DDC control systems exist, and a baseline system should be agreed upon by the park facilities and maintenance staff prior to controls upgrades. A complete control system upgrade would include existing HVAC systems and equipment serving the Arch, as well as systems and equipment serving periphery areas, including the Visitors Center. A control system upgrade of this magnitude is a large project and could be implemented in phases. This upgrade could result in reasonable increases of operability, maintainability, and efficiency of existing mechanical systems.

It is recommended that any and all future work associated with the HVAC, plumbing and fire sprinkler systems serving the Arch be thoroughly documented by the Park Service, or by contractors and consultants responsible for the work. It is important that the Park Service staff be able to reference this documented information for repair or replacement of any of the systems serving the Arch. This will also help to develop timeline of work performed to these systems which will aid in system evaluations in the future.

Electrical

Primary Power Distribution

The primary power distribution system will be maintained by the local power company.

Electrical Service

In the electrical service switchboard area, the HVAC control panel should be removed from the working space in front of the switchboard.

The lugs on the main switchboard should be re-torqued.

Emergency Power Distribution

The existing 235KW and 300 KW diesel powered generators should be replaced with two 350 KW diesel powered emergency generators. The generators should be periodically tested under load using the load bank.

Interior Lighting

Interior wiring conductors in feeders and branch circuits should be systematically replaced with new conductors with THWN/THHN insulation installed in existing conduits. A green insulated ground wire should be installed in all upgraded branch circuits and feeders. When feeder and branch circuit conductors are being replaced the associated conduit should be evaluated to determine where and if expansion fittings should be installed.

Grounding

When branch circuits and feeders are replaced or upgraded, a green insulated equipment grounding conductor should be installed.

Lightning Protection

The lightning protection system should be inspected and maintained by a company familiar with the Lightning Protection Institute standards such as *NFPA 780, Standard for the Installation of Lightning Protection Systems*.

Lighting

Existing lighting fixtures that still use incandescent lamps should be replaced with lighting fixtures that use fluorescent lamps or with new lighting fixtures that utilize LED lamps. Lighting controls should be updated to provide up-to-date automated controls.

Devices

The existing devices in the Arch legs should be removed and new devices provided with weatherproof cover plates to keep oily graphite out of the device contacts.

Exit Lighting

Exit lighting could be enhanced by adding additional exit lighting fixtures near the floor so that the exit lighting fixtures could be seen in smoke filled rooms. The incandescent illuminated exit lighting fixtures should be replaced with LED lamp type fixtures.

Emergency Lighting

Battery pack emergency lighting fixtures should be added along the stairwell in each of the Arch legs.

Fire Alarm System

Fire alarm speakers should be provided in the Arch legs. These speakers should be on a separate zone. These speakers will improve communication with people using the stairwell in the Arch legs during an emergency evacuation. Additional smoke detection could also be added in the Arch legs.

Intrusion Alarm System

Motion detectors should be added in strategic areas and connected to the intrusion alarm system. Water detectors should be installed on floors in area subject to water accumulation.

Communication System

Communication speakers and area of rescue assistance call stations should be added to the Arch legs.

Mechanical Equipment Connections

Consideration should be given to replacing existing motor starters with variable frequency drive controllers.

Tram Equipment Connections

National Park Service maintenance staff would like to have the two 125 HP tram motor generator sets replaced and relocated with solid state equipment. When the tram equipment is upgraded, the electrical wiring should also be upgraded to provide adequate clearance around all electrical equipment. The tram equipment should be powered from shunt-trip breakers. The shunt-trip breakers should be interlocked with the fire alarm system. If a fire alarm smoke detector in the tram equipment area detects smoke, the fire alarm system will signal the shunt trip breakers to open and shut off power to the tram equipment. This will reduce the spread of fire and smoke.

Plumbing

The existing sump pumps serving the Arch legs should be closely inspected when the trams are not in operation. Motor sizes should be recorded and pump capacity and sizes should be recorded if available. During the field investigation, it was not apparent that the sump pits are sealed. If they are not currently sealed, the pits should be provided with sealed or gasketed lids as required. The sealed sump pits should then be vented to the exterior. This will help to reduce the amount of moisture escaping the sump pits and migrating into the arch legs. This in turn may help to reduce high humidity levels that are at times present within the Arch legs. Continued maintenance of the pumps on an as-needed basis should be sufficient for preservation of the current sump pumps.

Code Compliance and Life Safety

It is acknowledged that it may not be practical or economically feasible to implement many of the changes that would be required to address the various code deficiencies noted and bring the Arch into full compliance with all current code provisions. Of the noted code deficiencies, smoke control is the most serious and should be given the highest consideration. However, any improvement that will enhance life safety should be considered.

Future treatment recommendations regarding exterior and interior accessibility of the Arch should be made in consultation with NPS Park Management and be in conformance with directives of the General Management Plan (GMP).

The following modifications are recommended for consideration:

1. In public areas and spaces through which visitors could be required to pass in an emergency, modifications to the stair treads and risers may be considered to improve the safety of users who may not be familiar with the existing conditions. It is recommended that these improvements be based on the International Building Code (IBC) Section 1009.3.
2. In public areas and spaces through which visitors could be required to pass in an emergency, modifications to handrails at stairs and ramps may be considered to improve the safety of users who may not be familiar with the existing conditions. It is recommended that these improvements be based on IBC Sections 1009.10 and 1012.
3. In public areas and spaces through which visitors could be required to pass in an emergency, modifications to guardrails at stairs and stair landings may be considered to improve the safety of users who may not be familiar with the existing conditions. It is recommended that these improvements be based on IBC Sections 1009.10 and 1012.
4. In public areas and spaces through which visitors could be required to pass in an emergency, modifications to ramps and landing may be considered to improve the safety of users who may not be familiar with the existing conditions. It is recommended that these improvements be based on IBC Sections 1008.1.4, 1008.1.5, and 1010.
5. By incorporating the following improvements, both legs of the Arch could be better equipped to allow for a safer means of visitor egress in the event of an electrical fire or tram malfunction.
 - a. Create four separate smoke tight compartments: the observation level, the electrical equipment area directly below the observation level, the north tram/stair leg, and the south tram/stair leg. This would involve sealing of all existing joints/gaps associated with the enclosure of the existing observation level, as well as installation of additional smoke partitions at each end of the observation level and at the equipment area below it to completely seal off one section of the Arch from the others. Refer to conceptual graphics in Appendix G. (This also assumes that fire rated and smoke tight door assemblies may be installed between the tram load zones at the base of the Arch legs and the main lobby of the visitor center.)
 - b. An investigation should be performed to evaluate the possibility of providing a fire suppression system to serve the mechanical and electrical equipment area below the observation level floor. This area has the highest probability for fire generation and propagation within the Arch itself. The possibility of providing a fire suppression system

to serve the observation level should also be researched. Possible suppression systems to serve these area include, but are not limited to:

- 1) Aqueous Film-Forming Foam (AFFF) systems
- 2) FM-200 suppression systems
- 3) Sapphire suppression systems
- 4) Water misting suppression systems

Further investigation and research would be required to determine an appropriate system type and means of integration into the spaces. Locations for piping, head or nozzles, and ancillary equipment and control panels would need to be researched and determined.

- c. The existing Simplex fire alarm system could be expanded to detect smoke in the Arch legs. Conventional smoke detectors would not operate efficiently in the Arch legs due to the presence of oily graphite in the air. Beam detector type smoke detectors located at several elevations in the Arch legs would operate more effectively.
- d. Fire alarm speakers could be added in the Arch legs, with each Arch leg on a separate zone. The existing Simplex fire alarm system could be expanded with additional amplifiers to handle the speakers. In the event of an evacuation down the Arch legs, pre-recorded messages can be announced through the speakers or a microphone at ground level can announce specific instructions through the speakers.
- e. Emergency breathing equipment could be provided within the Arch legs and/or the observation level for visitors and employees.
- f. The existing illumination level as well as energy conservation could be improved in the Arch legs by removing the incandescent lighting

fixtures and providing lighting fixtures with fluorescent lamps, or appropriate LED lamps if available. Lighting fixtures using metal halide, high pressure sodium, or mercury lamps would not be appropriate due to the fact that these lamps cannot restart immediately after a momentary power failure. The lighting fixtures in the Arch legs have backup emergency power from the emergency generator system. In the event of a power failure it takes approximately 10 seconds for the emergency generator to start and come on line. To a person in the Arch legs, 10 seconds without lighting would seem like a long time. It would be advisable to also add some emergency lighting fixtures with battery pack reserve power to bridge the time it takes for the generator to start up and come on line.

6. Existing finishes such as carpet that are routinely replaced by JNEM staff within the observation level of the Arch should be replaced with finishes that comply with current smoke development and flame spread requirements per IBC Chapter 8. This work would also expose all voids in the perimeter of this area and allow the entire observation level to be sealed "smoke tight" from the remainder of the other areas within the Arch.

Accessibility

The following improvements related to accessibility are recommended:

1. Provisions to accommodate accessible parking near the Arch monument entrances may be considered. (Refer to ABAAS F208 and F502.)
2. Modifications to facilitate an accessible exterior route connecting the immediate sidewalk area near the Arch bases outside of the building and leading to the main building entrances may be considered. (Refer to ABAAS F206 and F208.)
3. Modifications to the interior route leading to the tram load zone and the observation level at the top to the Arch are neither practical nor economically feasible. It is recommended that consideration be given to creating a “virtual experience” facility on the museum level that would allow disabled visitors to see and feel the same experience as those at the top of the Arch without physically being there.
4. Although not all disabled users will be able to access the observation level, the existing toilet room located adjacent to the observation level may be modified to meet current accessibility requirements. (Refer to ABAAS F213, 603, 604, 605, 606 and IBC Sections 1109.2.1 and 3409.9.4.)
5. Although the stairs within the Arch legs are part of an interior route that cannot be made fully accessible, modifications to the stair handrails may be considered to improve both life safety and accessibility. (Refer to ABAAS F210 and 504.)
6. Provisions for accessible signage along the exterior and interior accessible routes connecting the areas leading to the main entrances and tram load zones may be considered. (Refer to ABAAS F216.)

RECOMMENDATIONS FOR FURTHER RESEARCH

The following areas of study are recommended for further research or investigation:

1. Hazardous materials analysis.
2. Further investigation to determine causes and sources of observed active water leaks in the tram loading zone areas, apparently from either trench drains perpendicular to the ramps and/or expansion joints located where the ramps join the museum building.
3. Preparation of a Historic Structure Report, or an amendment to this report, to address the visitor center including identification of significant features and determination of the period of significance. Assessment of the visitor center was beyond the scope of this study. The park is currently in discussion with the Missouri SHPO regarding evaluation of the historic significance of the interior finishes and spaces. Findings of this evaluation may indicate that it would be appropriate to prepare an amendment to the National Historic Landmark documentation for the Gateway Arch to address the interior spaces and features.
4. Perform an additional HVAC study to investigate further the micro-climate within the Arch legs and the feasibility of HVAC system replacement or expansion in order to control or resolve the current moisture issues.
5. Prepare a fire safety study of the entire Arch complex.
6. Locate and obtain copies of the other structural studies listed in the bibliography of the December 1964 Bureau of Reclamation report, identified on pages 94 and 95 of this report.

Recommendations for Further Structural Investigations

Because the scope of testing, instrumentation, and inspection openings included as part of this Historic Structure Report was limited, the following recommendations are provided for further structural investigations. These additional tasks, including investigation openings and further monitoring and testing, coupled with the initial structural condition assessment of the Gateway Arch, review of previous of structural studies, original drawings and specifications, and the interviews conducted with various representatives of the design and construction teams, will help to ensure that a thorough structural evaluation is completed. The following additional studies are recommended:

1. Install data loggers in the north leg for one year for comparison between the two legs, as more evidence of moisture was observed in the south leg as compared with the north leg. As part of collecting the data, temperature readings of the interior skin should be taken using the infrared thermometer at various locations throughout the leg.
2. It is understood that as part of Phase II of the staining study, a more sophisticated monitoring system for temperature and relative humidity has been proposed. This monitoring system should include more monitors, as well as measuring the surface temperature of the interior plates on all sides of the legs as well as the air temperature—as included in the current limited monitoring system implemented as part of the Historic Structure Report study. As part of the long term monitoring, surface temperature of the interior plates as well as the air temperature should be measured. Measuring surface temperature of the interior plates will help to determine

the temperature variances within in each leg, thus providing a better understanding of how the stresses and strains vary between the different portions and faces of the interior skins of each leg. The thermal coefficient of expansion for A-7 carbon steel is a constant throughout the interior because all segments of the Arch are constructed of the same material; however, surface temperature is the variable, which contributes to differing internal stresses caused by varied expansion and contraction of the differing faces and segments of the Arch.

3. Observations of the interior of either or both legs of the Arch should be made when “rain” is occurring inside the Arch to gain a better understanding of this phenomenon and the actual temperature and relative humidity during the event. These visual observations should be correlated with temperature and relative humidity data collected at the same time.
4. Instrumentation should be installed at discrete, unobtrusive locations within the interior legs of the Arch. Strain gauges should be installed on all interior skin plates of the Arch at designated elevations, to evaluate the varied strains and stresses experienced by each portion of the legs, primarily as attributed to the varying temperatures caused by radiant heating of the sun.
5. Non-destructive evaluations of the hidden welds between the interior and exterior skins should be conducted in order to ensure that no significant section loss is occurring.
6. Close-up visual inspections should be made of exterior elements, including locations of the discolored field welds as well as staining marks caused by the bearing pads for the creeper derrick crane and rigging equipment. This further investigation will require special access

considerations; thus the feasibility of these inspections must be evaluated.

7. A further intrusive structural investigation should be conducted to verify existing concealed conditions as indicated in the structural drawings, reports, and specifications. This evaluation will include investigation openings, laboratory testing of materials for strength values, and further laboratory analysis.
 - Inspection openings should be made at the interior skin below the termination of the concrete. This includes coring from the interior at three to four locations to inspect the condition of the concrete and to determine its current moisture content. Ideally, cores should be taken in both legs in order to evaluate differences between the differences in construction between the north and south legs. One of the cores should be taken at a location where voids were experienced during the sounding inspections, in order to determine if free water is present. Inspection openings made below the termination of the concrete would also allow for an evaluation of the condition of the reinforcing bars and post-tensioning reinforcement as well as the carbon steel plates to determine extent of corrosion and section loss. Laboratory analysis of these cores should include a petrographic examination to evaluate conditions undetectable in a general visual examination and tests to evaluate the strength of the materials.
 - Inspection openings should be made at the interior skin above the termination of the concrete. This includes removal of a portion of the interior carbon steel plate at a few locations in each leg to inspect the condition of the interior and exterior plates as well as the stiffener angles and plates, and

fasteners and tie rods, to determine extent of corrosion and section loss. The following types of samples should be removed at these inspection openings for laboratory analysis:

- Samples of the isolated diagonal tie rods and fasteners, including locations with and without surface corrosion, to determine strength values, as well as to evaluate the effect of corrosion on the strength of these elements.
 - Samples of the interior skin plates and stiffener angles for a metallurgical analysis, including chemical composition and determination of yield and ultimate strengths.
 - Inspection openings should be made from the interior at locations of the reinforced concrete foundation walls. This includes coring at locations in the concrete foundation walls beneath both legs to inspect the condition of the concrete and reinforcement. The current moisture content should be determined and the differences between the legs evaluated. Laboratory analysis of these cores should include a petrographic examination to evaluate conditions undetectable in a general visual examination and compressive tests to evaluate the strength of the concrete cores.
8. Further investigation of the streaking observed on the steel should be conducted to attempt to correlate interior with exterior conditions, as well as to coordinate with information obtained by review of the original drawings, specifications, previous reports, and correspondence.
9. Further structural analysis, including computer modeling or structural calculations, should be performed to

evaluate the stability of the Arch in relation to withstanding the current design load criteria for wind and seismic considerations.

COST ESTIMATES

Class C Cost estimates for the recommended work are provided on the following pages.

Class C Construction Cost Estimate
Jefferson National Expansion Memorial Gateway Arch

Estimate By: Dennis Sieh/Alvines
Date: 10/28/2009

St. Louis, MO

Reviewed By: DW/MG/DS/SK
Date: 10/28/2009

Estimate is Based on 2009 Costs

Bid Item	Recommended Markup	Building Cost Costs	Site Costs	Combined Building and Site
General		\$840,190	\$0	\$840,190
Mechanical		\$370,000	\$0	\$370,000
Electrical		\$794,000	\$0	\$794,000
Subtotal Direct Construction Cost		\$2,004,190	\$0	\$2,004,190.00
Published Location Factor (0 Percent)	0%	\$0	\$0	\$0
Remoteness Factor (0 Percent)	0%	\$0	\$0	\$0
Federal Wage Rate Factor (3 Percent on labor portion)	3%	\$30,063	\$0	\$30,063
Design Contingency (15 Percent)	15%	\$300,629	\$0	\$300,629
Total Direct Construction Costs		\$2,334,881	\$0	\$2,334,881
Standard General Conditions (5 Percent)	5%	\$116,744	\$0	\$116,744
Government General Conditions (5 Percent)	5%	\$116,744	\$0	\$116,744
Historic Preservation Factor (5 Percent)	5%	\$116,744	\$0	\$116,744
Subtotal NET Construction Cost		\$2,685,114	\$0	\$2,685,114
Overhead (10 Percent)	10%	\$268,511	\$0	\$268,511
Profit (10 Percent)	10%	\$268,511	\$0	\$268,511
Subtotal NET Construction Cost		\$3,222,136	\$0	\$3,222,136
Contracting Method Adjustment (8a)	0%	\$0	\$0	\$0
Estimated NET Construction Cost		\$3,222,136	\$0	\$3,222,136
TOTAL Estimated NET Cost of Construction (Inflated to Spring 2011)	5%	\$3,383,243	\$0	\$3,383,243
Construction Contingency (0%)	0%	\$0	\$0	\$0
Construction Management (NAPA 0%)	0%	\$0	\$0	\$0
GROSS CONSTRUCTION COSTS		\$3,383,243	\$0	\$3,383,243

**National Park Service
Jefferson National Expansion - Gateway Arch Renovation
St. Louis, Missouri**

Schematic Estimate

BCC Job No.: 09-10-0139

October 29, 2009



Schematic Estimate	BCC Building Cost Consultants, Inc. National Park Service Jefferson National Expansion - Gateway Arch Renovation St. Louis, Missouri 10/29/09 BCC Job No.: 09-10-0139	QTY. NO. UNITS	QTY. UNIT	MATERIAL & LABOR PER UNIT	MATERIAL & LABOR TOTAL
ITEM	DESCRIPTION				
FINAL SUMMARY SHEET					
	Smoke Compartmentalization at Top of the Arch (Page 2)				\$57,700.00
	Patching Associated with Installation of Fire Suppression System at Observation Level (Page 3)				9,760.00
	Testing Allowance/Further Investigations for Exterior Skin of Arch (Page 4)				437,500.00
	Stainless Steel Eyelet Hooks to Provide Rigging Supports for Accessibility to Upper Levels of Arch (Page 5)				86,630.00
	Removal of Floor Infills at the Lower Levels of the Arch Returning the Space to As Originally Designed (Page 6)				58,160.00
	Testing and General Maintenance to Concrete Walls in Lower Level (Page 7)				163,630.00
	Repair Trench Drains at Entry Ramps (Page 8)				22,880.00
	Replace Folding Door at Emergency Restroom Located at the Observation Level (Page 9)				3,930.00
	PROJECT CONSTRUCTION TOTAL =				\$840,190.00
NOTE:	The following mark-ups are included in the above costs:				
	Subcontractor Overhead, Profit, Insurance and Bond -	10%			
	Design Contingency -	0%			
	Escalation -	0%			
	Material Access / Phasing /Protection-	25%			
QUALIFICATIONS					
1	No sales tax is included. Assumed facility is tax exempt.				
2	No asbestos removal is included.				
3	No costs are included for furniture, furnishings or movable equipment.				
4	Assumed construction to be phased and partial hour work.				
5	Assumed project to be competitively bid.				
6	The construction costs shall be used for budgeting and planning purposes only and shall not be used as an actual bid as given by a contractor to build the project.				
7	The construction totals are rounded to the nearest \$10.00.				

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

Class 'C' Cost Estimate



1800 O Street, Ste. 104, Lincoln, Nebraska 68508 (402) 477-6161

October 19, 2009

Gateway Arch - Historic Structures Report

Alvine No. 2008 0399

Electrical/Mechanical Improvements Recommendations – Class 'C' Cost Estimate

Electrical Service

Retorque lugs on switchboard: \$5,000.00.

Interior Wiring

Replace existing interior branch circuit and feeder conductors add expansion fittings and add grounding conductors: \$500,000.00.

Lighting

Replace existing incandescent lighting fixtures with new fluorescent or LED lighting fixtures: \$40,000.00
New automated lighting controls: \$20,000.00.

Devices

Replace existing devices with new devices: \$6,000.00.

Exit Lighting

Add additional exit lights near floor: \$5,000.00.
Replace existing incandescent exit lights with LED exit lights: \$1,000.00.

Emergency Lighting

Add emergency lights in the Arch stairwells: \$35,000.00.

Fire Alarm System

Add fire alarm speakers in the Arch Legs: \$20,000.00.
Add Fire alarm smoke detectors in the Arch Legs: \$15,000.00

Intrusion Alarm System

Add motion detectors: \$5,000.00.
Add water detectors: \$2,000.00.

Communication System

Add area of rescue call stations and speakers: \$15,000.00.

Mechanical Equipment Connections

Replace existing motor starters with variable frequency drives: \$75,000.00.

Tram Equipment Connection

Upgrade electrical power distribution to Tram equipment when the two 125 HP motor generator sets are replaced and relocated: \$50,000.00 (Feeder cost included under Interior Wiring).

Control System Upgrade/Replacement

Demolition of existing pneumatic control components and panels; upgrade/expansion of DDC control system serving Arch system: \$220,000.00

Chilled Water, Steam, and Condensate Piping Insulation

Removal of existing piping insulation and replacement with fiberglass piping insulation: \$50,000.00.

Observation Deck Fire Suppression

Water misting fire suppression system: \$60,000.00

Wet chemical fire suppression system: \$75,000.00

Observation Deck Breathing Apparatus

Add breathing apparatus for use by occupants at the observation deck: \$25,000.00

BIBLIOGRAPHY

Ayres, Janet, and Jennifer Rawlings. *Finding Aid to the Jefferson National Expansion Memorial Research Reports*. St. Louis, Missouri: JNEM, March 2001.

The authors, researchers at the Jefferson National Expansion Memorial archives, provide a thorough inventory of the historical reports and research conducted on subjects related to the Jefferson National Expansion Memorial and territorial expansion. Source: JNEM archives.

Bahr Vermeer Haecker Architects and Wiss, Janney, Elstner Associates, Inc. *Gateway Arch: Corrosion Investigation Part I*. St. Louis, Missouri: JNEM National Park Service, May, 2005.

The investigation seeks to determine the cause of corrosion in the carbon steel and stainless steel skins of the Arch. The report noted that staining and corrosion were observed on the carbon steel and stainless steel skins. Micro-climate conditions, galvanic corrosion, and deterioration of welds were cited as potential causes of corrosion. The findings of this investigation are discussed in the current Historic Structure Report. A second phase of the corrosion (staining) study is planned. Source: Wiss, Janney, Elstner Associates archives

Bellavia, Regina M., and Gregg Bleam. *Cultural Landscape Report*. Omaha, Nebraska: National Park Service, 1996.

The report provides a comprehensive history of the designed landscape, assesses existing landscape and site conditions, identifies significant landscape features, and provides recommendations for treatment. Source: JNEM website

Black and Veatch Special Projects Corps. *Vulnerability Assessment: Jefferson National Expansion Memorial St. Louis, Missouri*. St. Louis: National Park Service Jefferson National Expansion Memorial, February 13, 2003.

The structure of the Arch was studied in reference to determining its ability to withstand a sudden impact. Recommendations for retrofits were made based on the findings. Further details are not included in this study because of the sensitive nature of the material. Source: JNEM confidential archives

Brown, Sharon A. *Administrative History Jefferson National Expansion Memorial National Historic Site*. Washington, D.C: National Park Service, June 1984.

The author, a researcher for the JNEM, gives a descriptive history of the development of the National Park from 1933–1980. The report provides a detailed, chronological synopsis of the legislative, political, and economic issues surrounding the park. Source: National Park Service online archives

Cayes, Lloyd R., and Charles F. Scheffey. Memorandum from the Bureau of Reclamation, excerpts of conclusion from *Aerodynamic Stability of the Jefferson National Expansion Memorial Gateway Arch*. Denver, Colorado: Department of the Interior, July 1965.

Official Report of Proceedings before the U.S. Department of Interior, Bureau of Reclamation on the Structural Aspects of the Gateway Arch, St. Louis, Missouri. Transcript. St. Louis: February 3, 1965.

Capps, Michael. "Jefferson National Expansion Memorial, Architect Eero Saarinen." 2006.
<http://www.nps.gov/jeff/planyourvisit/architect.htm>. Accessed on October 23, 2008.

The author, a researcher for the JNEM, provides an abbreviated history of Eero Saarinen including a description of his childhood, design of the Arch, and legacy. The paper includes a brief chronology of significant events in Eero Saarinen's life. Source: JNEM archives

Cayes, Lloyd, and Charles F. Scheffey. *Aerodynamic Stability of Jefferson National Expansion Memorial Gateway Arch*. Washington, D.C: U.S. Department of Commerce, Bureau of Public Roads, July 1965.

The report was published in response to a request from the Bureau of Reclamation that wind tunnel tests be done to determine the aerodynamic stability of the Arch. The Bureau of Public Roads undertook the study based on their experience and test methodologies used with suspension bridges. The report outlines the test methods and describes results, and concludes that high winds could severely compromise the structural stability of the Arch. (Other documents cited in the Historic Structure Report describe responses to the aerodynamic stability assessment, including criticism the report received for its conservative assumptions and lack of understanding of the structural system of the Arch.) Source: JNEM archives

Daly, Leo A., and Cermak, Peterka, and Petersen (CPP), *Blast Assessment and HVAC Study for the Jefferson National Expansion Memorial, St. Louis, Missouri*. Denver, Colorado: National Park Service, Denver Service Center, December 14, 2007.

The HVAC system of the Arch was studied in reference to determining its ability to withstand tampering and infiltration. Recommendations for retrofits were made based on the findings. Further details are not included in this study because of the sensitive nature of the material. Source: JNEM confidential archives.

Daniels, Rudolph. *Trains Across the Continent, North American Railroad History*, second ed. Bloomington, Indiana: University of Indiana, 2001.

The publication provides a historical background for St. Louis as it developed into an epicenter for railroad traffic and reaffirmed its position as the Gateway to the West.

Detmers, Bruce, Architect, with Eero Saarinen and Associates. Interview by Deborah Slaton and Michael Ford, Wiss, Janney, Elstner Associates, Inc., April 1, 2009.

The oral history interview provides insight in the construction of the Arch as told by Bruce Detmers, Project Architect for Eero Saarinen and Associates, to members of the Historic Structure Report study team. The interview was conducted at the Yale University Archives after

reviewing the Saarinen Collection. The interview provides insight into the latter stages of the Gateway Arch design, issues over post-tensioning, the condition of the stainless steel skin, and the personalities involved. Source: WJE archives

Eero Saarinen and Associates, original construction documents, October 16, 1961.

The detailed set of documents provides information for the structural design and development of the Arch. Source: JNEM archives

———. *Gateway Arch and Visitors Center specifications*. Denver, Colorado: Jefferson National Expansion Memorial [JNEM] National Park Service MacDonald Collection, November 10, 1961.

The specifications are a guide to materials and construction methods originally outlined by Eero Saarinen and Associates for construction of the Arch. Source: JNEM archives

———. Specification Addendum No. 1. St. Louis, Missouri: JNEM MacDonald Collection, November 18, 1961.

Addendum No. 1 was issued during the bidding process and was accompanied by a drawing. The addendum clarified some of the language in the specifications and answered questions brought forth by contractors including those pertaining to the foundation and material specifications. Source: JNEM archives

———. Specification Addendum No. 2. St. Louis, Missouri: JNEM MacDonald Collection, November 30, 1961.

Addendum No. 2 was issued during the bidding process and was accompanied by a drawing. The addendum clarified some of the language in the specifications and answered questions brought forth by contractors including questions about weld specifications. Source: JNEM archives

———. Specification Addendum No. 3. St. Louis, Missouri: JNEM MacDonald Collection, December 9, 1961).

Addendum No. 3 was issued during the bidding process and was accompanied by numerous drawings. The addendum offered additional weld techniques and noted changes to the construction drawings. Source: JNEM archives

———. Specification Addendum No. 4. St. Louis, Missouri: JNEM MacDonald Collection, December 11, 1961.

Addendum No. 4 was issued during the bidding process and outlined the wage rates. The addendum was deleted when new wage rates were implemented. Source: JNEM archives

———. Specification Addendum No. 5. St. Louis, Missouri: JNEM MacDonald Collection, January 3, 1962.

Addendum No. 5 was issued during the bidding process and was accompanied by numerous drawings. The addendum clarified the location of joints and framing members on the Arch, identified a method for determining acceptable flatness of the Arch outer skin, revised the foundation plan, and made changes to the construction documents. Source JNEM archives

———. Specification Addendum No. 6. St. Louis, Missouri: JNEM MacDonald Collection, January 13, 1962.

Addendum No. 6 was issued during the bidding process and aimed to clarify issues regarding the structure and final cleaning of the Arch. Source: JNEM archives

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APPENDICES

Appendix A – Copies of Selected Archival Documentation

Appendix B – Finishes Analysis

Appendix C – Previous Structural Studies

Appendix D – Interior Temperature and Relative Humidity Monitoring

Appendix E – Install New Entrance Ramps to Arch Monument and Visitor Center, PMIS 150546

Appendix F – Base Drawings showing Existing Conditions and Conceptual Smoke Separation Graphics

Appendix G – Exterior Condition Assessment Drawings

Appendix H – Oral History Transcripts

Appendices included in Volume 2

Appendix I – Selected Structural Drawings

Appendix J – Scanned Original Drawings

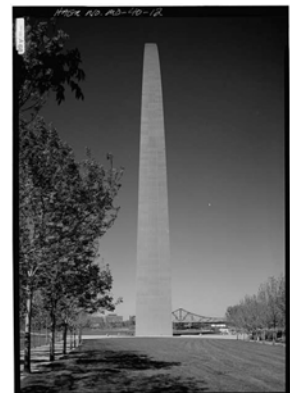
Appendix K – Scanned Original Shop Drawings

Appendix L – Other Drawings

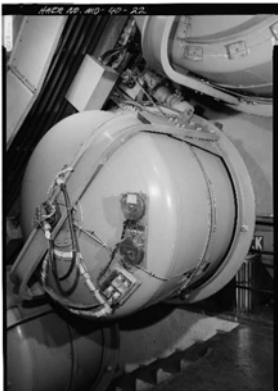
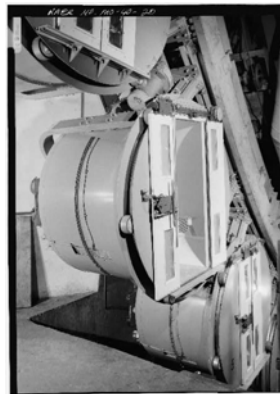
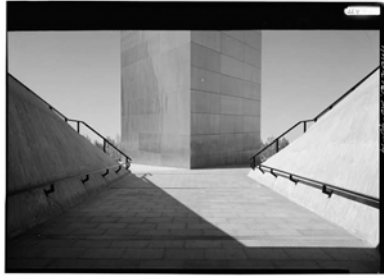
Appendix A – Copies of Selected Archival Documentation

- HAER MO-40, 1978
- Yale University Library, Manuscript Group 593, *Eero Saarinen Collection*.
- *Richard Bowser Papers, 1958–1982*, Jefferson National Expansion Memorial archives, *Richard Bowser Papers, 1958–1982*.
- Various sources. Archival photographs provided by the following sources:
 - Ken Kolkmeier
 - National Park Service Archives
 - Bruce Detmers
- Gateway Arch and Visitor Center Specifications, *Section 4: Metal Arch Shell*, November 10, 1961

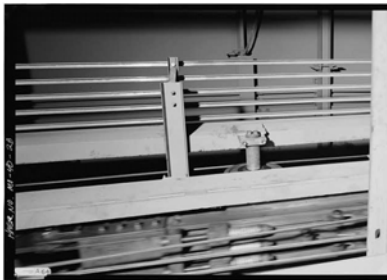
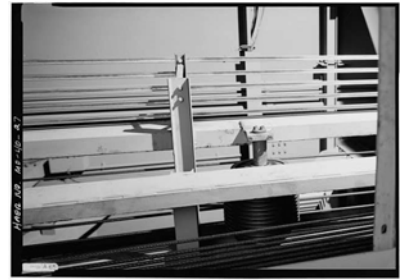
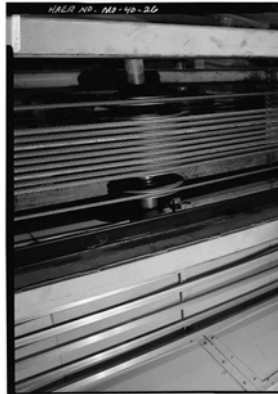
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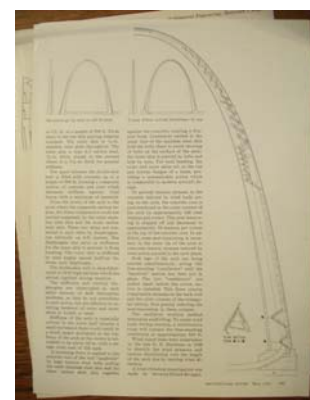
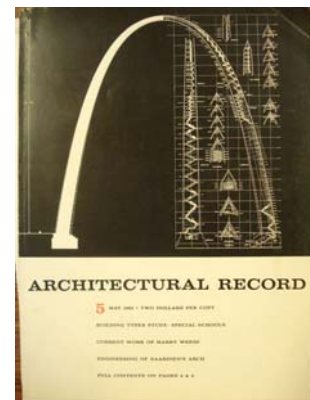
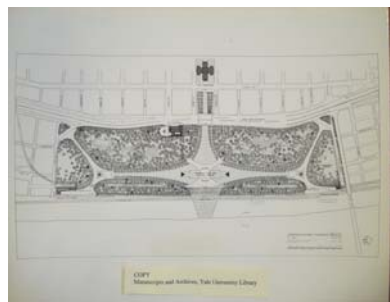
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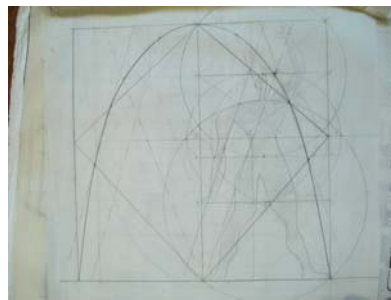
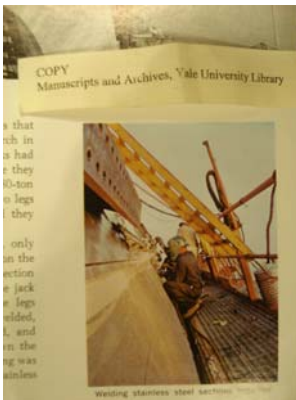
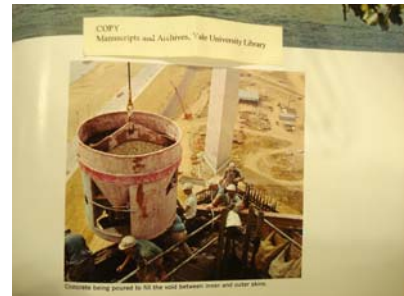
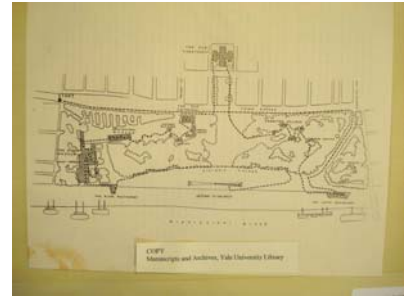
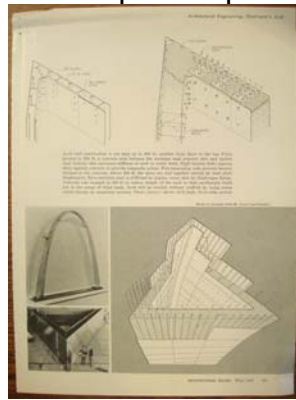
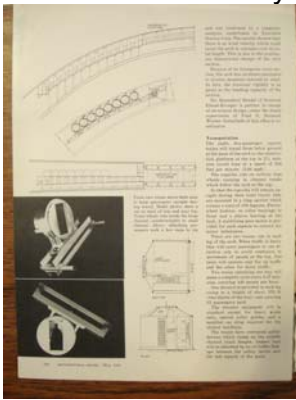
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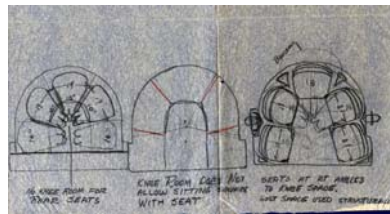
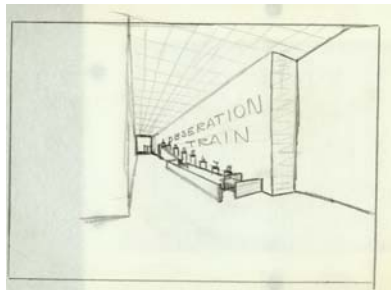
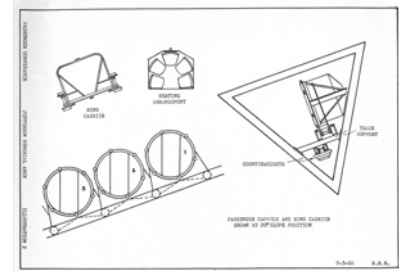


Source: Yale University Library, Manuscript Group 593, Eero Saarinen Collection.

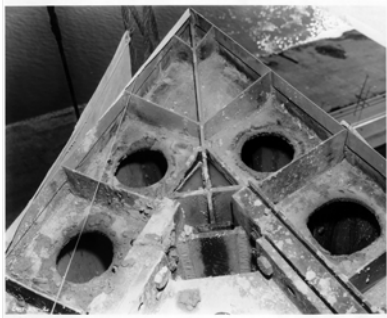


Source: Yale University Library, Manuscript Group 593, Eero Saarinen Collection.

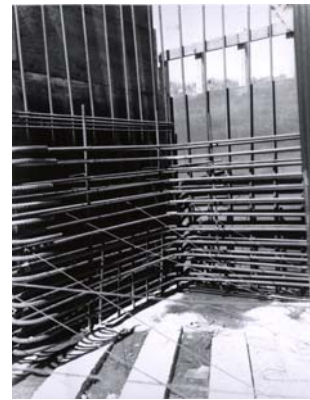




Source: Ken Kolkmeier



Source: National Park Service Archives



Source: Bruce Detmers



SECTION 4

METAL ARCH SHELL

SCOPE

4-1.1 WORK INCLUDED: This section pertains to exterior and interior metal plates and plate assemblies, and related connections, forming outer and inner skins of shell of arch, furnished and erected in conformity with specifications and drawings.

General requirements which shall control construction of arch structure proper, are specified in Section 2, ARCH STRUCTURE - GENERAL, which shall be supplementary to, and shall form a part of this Section 4.

4-1.2 ALTERNATE: An alternate dollar amount shall be stated in the Bid Form covering the substitution of composite steel facing plates consisting of stainless steel cladding on carbon steel backing plate, in lieu of solid stainless steel facing plates, as specified, for outer skin of arch shell.

It is the intention of the Government to award the contract on the basis of using solid stainless steel as the covering for the Gateway Arch, UNLESS the cost of stainless clad steel is substantially lower so that it would be in the interest of the Government, as determined by the Contracting Officer, to purchase stainless clad steel.

GENERAL REQUIREMENTS

4-2.1 QUALITY OF WORK. Materials incorporated in the work and workmanship shall be subject (1) to approval of Contracting Officer, and (2) to inspection, sampling and testing by Contracting Officer, as set forth in GENERAL and SPECIAL PROVISIONS.

4-2.2 WELDING: All welding shall be continuous and shall develop not less than the full strength of the welded components in shear and moment. All seams in both the outer and inner skins shall be full welded; all seams in outer skins shall be water-and-weather-tight.

4-2.3 BASIC DESIGN: The completed arch shall conform in all respects with the aesthetic design, geometric configurations, and structural design as delineated on the contract drawings; no deviations therefrom, except for such permissible variations as are allowed within erection tolerances established as set forth in Section 2, shall be made by the Contractor unless prior specific written approval has been obtained from the Contracting Officer.

Surfaces of exterior and interior skins shall be faired from plate to plate and from segment to segment, free from individual or cumulative deformations or variations in excess of established erection tolerances. Required shapes, straight-line and developed dimensions, and formed warpage and curvature of individual plates, and required location and alignment of seams between plates, shall conform to geometry and patterns indicated by contract drawings and approved shop drawings.

4-2.4 COORDINATION: The complete coordination of fabrication, pre-assembly, delivery and erection of the metal arch shell, in accordance with progress of general construction of the project, shall be the full responsibility of the Contractor.

4-2.5 MATERIAL HANDLING, DELIVERING, STORING: All materials shall be handled, shipped, moved, stored, and effectively protected at all times during fabrication, and during and subsequent to erection in such manner that shapes, surfaces (finished or unfinished) and integrity of respective members and components are completely safeguarded against damage or defacement. Any material which is damaged or defaced shall be subject to rejection by the Contracting Officer and replacement at Contractor's own expense.

The provision and maintenance of effective and adequate protective measures and means to prevent damage and defacement of any kind to finished surfaces of stainless steel shall be full responsibility of the Contractor.

4-2.6 HOLES AND OPENINGS: All cutting of work furnished and installed under this Section shall be done by Contractor to accommodate work of other trades by shop cutting in accordance with approved shop drawings and by field cutting; no field cutting for such purposes shall be done without prior approval of Contracting Officer.

4-2.7 INSPECTION: Contracting Officer shall be permitted to inspect all mill production, shop fabrication and field erection. Contractor shall furnish Contracting Officer with notices of, and make necessary arrangements for access to all work at mill or shop.

MATERIALS

4-3.1 OUTER SKIN PLATES:

a. Base Bid: Under Base Bid, plates for outer skin shall be 3/8 inch thick and 1/4 inch thick, solid stainless steel plates, as shown on drawings, conforming to ASTM A-167, Grade 3 (Type 304 containing min. 18% Chromium and min. 8% Nickel), finished with ~~special No. 100 grit~~ finish on external face to match approved sample. #3

1. Permissible Variation in Thickness and Weight: No stainless steel plate shall be more than 0.01" under specified thickness. Overweight per lot (as defined in ASTM A-6) shall not exceed 12% for 1/4 inch, nor 9% for 3/8 inch thick stainless steel plates; overweight of individual plates shall not exceed 1-1/3 times amounts specified above.

b. Alternate Bid:

1. Under alternate bid, if accepted, plates for outer skin shall be 3/8 inch thick and 1/4 inch thick, rolled, annealed and pickled composite steel plates, consisting of stainless steel cladding plate. Stainless steel cladding shall be Type 304 (18-8), with a nominal cladding thickness of 0.050 (minus 2% to plus 5% thickness of cladding).

2. Carbon steel backing plate shall
External surface of

Stretching Leveling: Prior to shipment from mill to fabricator.

c. Flatness in Width of Outer Plates: When the flatness across the width of plates deviates more than $1/8"$ from table-top flatness, the deviation shall be corrected to $1/8"$ or less.

d. Protection shall be provided for all stainless steel by manufacturer, of proper type to adequately and effectively prevent damage and defacement of stainless steel during subsequent fabrication and erection operations. Refer to Article 4-2.5.

4-3.2 INNER SKIN PLATES Inner skin plates shall be solid carbon steel plates conforming to ASTM A-373, of thickness indicated on the Drawings. Plates shall have smooth clean surfaces, and no carbon steel plates shall be more than 0.01" under specified thickness. Overweights shall not exceed percentages set forth in ASTM A-6. Prior to shipment from mill to fabricator, plates shall be tested and variations of any plate in excess of one-half the permissible variations from flatness as established in ASTM-A6, Table XI, shall be rectified.

4-3.4 HIGH TENSION BOLT STUDS, including nuts, washers, and connectors therefor, shall comply with the requirements of ASTM A-325. Use and installation of these bolts shall conform to requirements of Specifications for Structural Joints Using A325 bolts, as approved by Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, March, 1960, and endorsed by American Institute of Steel Construction and Industrial Fasteners Institute.

NOTE: All bolts installed in portion of arch shell with concrete core, as indicated on drawings, shall be coated with an approved agent which shall break all bond between bolts and concrete; all bolts and connectors at the 3 corners of the arch shell shall be wrapped or padded in approved manner so that movement of connectors is not impeded by concrete when bolt assemblies are tightened.

4-3.5 MILD-STEEL BOLTS AND STUDS, including nuts and washers therefor, shall conform to ASTM A-307. For sizes and nut and washer requirements refer to drawings.

4-3.6 PAINT for shop painting and field touch-up, shall be approved red-lead-iron-oxide metal primer conforming to either Steel Structures Painting Council Specifications for Paint #2 (linseed oil and alkyd varnish vehicle) or Paint #3 (fractionated linseed oil vehicle).

APPROVALS

4-4.1 MATERIALS: Prior to ordering materials and starting work, Contractor shall submit written request for approval of Contracting Officer, stating sources, makes, brands, types, properties and specification details, of all materials which he proposes to incorporate in the work. Samples of materials shall be submitted for approval as requested by Contracting Officer.

4-4.2 SHOP DRAWINGS: Completely detailed shop drawings shall be submitted for approval of Contracting Officer, in accordance with Article 27 of GENERAL PROVISIONS, showing layouts and details for various parts of work; thicknesses, alloys, finishes of metals; all dimensions and all radii of curvature and warpage of all plates, details of connections, field measurements, and related data. No material shall be fabricated prior to approval of shop drawings.

Erection drawings shall feature erection marks referring to respective details on shop drawings. They shall also show any and all temporary attachments proposed for purposes of assembly or erection.

Shop drawings shall be completely correlated with shop drawings of work of other trades as required under Section 2, ARCH STRUCTURE - GENERAL.

4-4.3 FULL-SIZE MOCK-UP: All outer and inner skin plates, all structural members and connections of a permanent or temporary nature required for erection of the mock-up specified in Par. 2-2.12 of Section 2, shall be furnished and erected. Approval of the mock-up structure shall form the basis for all further fabrication and erection of actual arch structure in place.

FABRICATION

4-5.1 WORKMANSHIP: Work shall be fabricated in strict accordance with drawings and details, and with approved shop drawings. Jointing and inter-sections of plates and component parts at seams and joints shall be accurately and uniformly made and fitted true to line, width, and alignment, as indicated. External faces of adjoining plates shall be flush on both sides of seams. All components shall be accurately cut and formed to required shapes and sizes. Completed plates, and completed assemblies of plates for outer and inner skins, shall be free from bulges, depressions, buckle, tool marks and similar defects.

4-5.2 PRE-ASSEMBLY: Work shall be pre-assembled in shop to the maximum extent compatible with transportation and erection procedures; and, insofar as practicable, fitting and matching, and adjustments of individual parts and contacting faces and edges of mating assemblies shall be carried out in shop prior to shipment to assure required, acceptable fit during erection.

4-5.3 CUTTING: Rolled structural sections shall be cut by standard methods to produce accurate and proper fit.

a. Stainless Steel Plates (either solid or composite, as case may be) shall be cut by shearing, and all edges, arrises and bevels shall be ground to required lines and crosssections in preparation for welding processes and to produce required fit at joints and seams.

b. Carbon Steel Plates (except composite plates) may be cut by standard methods of shearing and flame-cutting. Edges shall be ground in preparation for welding and to produce required fit at joints and seams.

4-5.4 FORMING: All plates of outer and inner skins shall be cold-formed accurately to the required surface convexity, concavity or warp in

accordance with dimensions and radii indicated by drawings and established on approved shop drawings. Stiffener angles and channels shall be cambered or cut to conform with required warpage or curvature of plates.

"4-5.5 TOLERANCES. Maximum permissible variation in dimension of any true or developed dimension from corner to corner on perimeter of individual plates shall be limited to a minus tolerance of 3/16-inch and diagonal dimension (true or developed) between opposite corners shall be limited to a minus tolerance of 1/4 inch. No plus tolerances will be allowed."

4-5.6 SHOP WELDING: All welding in shop assemblies shall be executed by the same means and under the same qualifications as specified in Articles 4-6.1 through 4-6.6.

4-5.7 SHOP CLEANING- SHOP PAINTING- ETC.:

a. Cleaning: Upon completion of shop work, any and all parts shall be cleaned.

1. Carbon Steel shall be shop cleaned to remove oil, grease, dirt, loose mill scale, loose rust, stratified rust, weld flux and weld spatter, and other foreign matter by methods specified in Steel Structures Painting Council Specifications Nos. SP-1, Solvent Cleaning, and SP-2, Hand Cleaning, all as set forth in Article 6-5.9 of Section 6.

2. Stainless Steel shall be shop cleaned to remove dust, dirt, stains and other foreign matter, employing such methods as are particularly recommended in writing by manufacturer of stainless steel employed in work, and approved on trial basis by Contracting Officer. Immediately after such cleaning, protective coverings shall be restored, and any and all unprotected stainless steel surfaces given adequate protection. Refer to Article 4-2.5.

b. Shop Painting: With exception of stainless steel, and with the exception of (1) surfaces contacting or embedded in concrete and (2) surfaces within 3 inches of welds, all carbon steel parts and components shall be shop painted at least one full-covering coat of specified paint, as set forth in Article 6-5.9 of Section 6, STRUCTURAL STEEL.

All carbon steel parts and components forming and within cavities between inner and outer skins of arch shell above Station 41 (concrete core) shall be given not less than two coats of shop paint.

WELDING

4-6.1 INSPECTION: Determination of acceptability

"The strength and rigidity of the joints shall be equal to that of the plates being joined. Soundness of the weld shall meet or exceed the requirements of the ASME Boiler Code, Section 8."

extent or defective weldment.

Any portion of a weld having defects equal to or greater than 2 percent of thickness of the metal x 1/4" in length shall be replaced. The

aggregate total area of all other defects in any one flange or plate weldment shall not exceed 5 percent of respective area; and any portion of a joint weldment having defects in excess of this amount shall be replaced.

Contractor shall pay for removal of all defective welds and replacement with non-defective weldment.

a. Radiographic Tests, whenever employed as specified above, shall be made in conformance with the requirements of ASTM E-94, Recommended Practice for Radiographic Testing. The radiograph shall be obtained by placing the film as close to weld surfaces as practicable and exposing it by a technique which will determine quantitatively the size of defects with thickness equal to or greater than 2 percent of the thickness of the base metal.

The Laboratory's radiographic test reports shall include:

- (1) Identification of parts tested.
- (2) Radiographic test job number.
- (3) Findings and location of all defective areas which equal or exceed 2 percent of the thickness of the base metal x 1/4" in length.
- (4) Findings and location of defects when total of all inclusions in weld equal or exceed 5 percent of the weldment.

Defective areas reported under (3) and (4) above shall be logged directly on film or a print so that defective area can be located.

4-6.2 WELDING RODS: Except where otherwise specified or noted, welding rods shall conform to requirements for one of the Class E60 series electrodes of Tentative Specifications for Mild Steel Arc-Welding Electrodes, AWS A5-1, ASTM A-233, and shall be used only with type of current, polarity and in positions permitted by above named specifications.

4-6.3 WELDING PROCEDURES: All welds, except as otherwise specified or noted, shall be shielded arc fusion welding made in accordance with requirements of Standard Code for Arc and Gas Welding in Building Construction of the American Welding Society. Automatic submerged arc welding will be permitted if performed in strict accordance with A.W.S. "Standard Specifications for Welded Highway and Railroad Bridges". Welding power shall be sufficient to obtain complete fusion without burn-thru or distortion.

All welding operators shall be previously qualified by tests prescribed in the American Welding Society's Standard Qualification Procedure, or by such other tests as Contracting Officer may require. Welds shall be made only by operators who are qualified to perform the type of work required, as evidenced by their passing required tests. Tests completed more than six months prior to employment on this job will not be acceptable. Contractor shall assume all costs incident to procedure and operation qualification on tests and subsequent qualifying tests required by Contracting Officer.

4-6.5 b. ADDITIONAL OPTIONAL WELDING METHOD. Contractor shall have option of employing the following method of joining outer skin plates (either solid stainless steel or composite steel plates, as the case may be). However, only the one method selected by Contractor shall be used for joining all outer skin plates. 400

The welding procedure shall involve a pass from the outside face and a pass or passes from the inside face, using a gas shielded arc process as qualified below. The outside weld shall have a smooth, uniform appearance. It shall be free of undercut and prominent rippling. The weld shall be $3/8$ " wide with a minus tolerance of $1/8$ ". The weld shall have a maximum height above the surface of the base metal of $3/32$ ". Above Station 53 the width of the weld may be increased, if necessary, to make adjustment for plate joints.

The electrode or filler metal wire for all welding shall be austenitic stainless steel type 308, 309, or 310 of such composition as to result in crack-free welds.

For out-of-position (field) welds, the following procedure is desired:

1. "Short-arc", "dip-transfer" or equivalent method for outside face weld, using AWS-ASTM-ER 308 wire of .035" diameter and appropriate mixture of carbon dioxide argon and/or helium shielding gas. The root-gap at the outer face of the plate shall be $1/16$ " minimum with $1/32$ " minimum root face and bevel towards inner face commensurate with respective position.

2. Outside weld shall be made first as vertical passes to progress from top to bottom.

3. For inside welds, root of outside bead is to be conditioned by careful grinding or chipping; then weld is to be completed by gas-shielded arc process, using specified type electrodes or similar wire.

Incomplete fusion between outside bead and inside weld, of length limited by boiler code Section 8, shall not exceed 10% of the plate thickness under any circumstances. The entire method of joining shall be subject to approval by Contracting Officer, based on test welds made on trial basis.

11. Add a new sub-paragraph, designated sub-paragraph 4-6.5.c, reading as follows:

4-6.5.c. OTHER WELDING METHODS. The Contractor, subject to approval of the Contracting Officer, may utilize any other satisfactory welding method that will achieve results specified.

4-6.4 EB WELD INSERT PROCEDURE: All welds connecting stainless steel plates (either solid stainless steel or composite steel plates, as case may be) of outer skin, horizontal as well as vertical, and both shop and field welded, shall be made by the consumable insert method of root pass welding. The process involves the use of an EB Weld Insert which is completely fused by a conventional inert-gas-shielded tungsten arc torch. Through use of this method, welding from the inside of the outer skin shall result in the deposition of a root pass which is uniform and smooth on the outside face of the stainless steel plate assembly.

Type "A" insert material of 308L composition and 5/32" nominal size shall be employed in standard coils, for root pass weld. After inspection and approval of the root pass, completion of the weld by use of appropriate filler material and standard welding process, ~~all as specified hereinbefore~~, shall be performed.

Root pass welding by use of EB Weld Insert method shall be executed in strictest conformity with procedure developed by prior trial runs made under observation by the Contracting Officer, whose approval of method and welding personnel is required.

4-6.5 OPTIONAL WELDING METHOD: Contractor shall have option of employing following method of joining outer skin plates (either solid stainless steel or composite steel plates, as case may be). However, only the one method as selected by the Contractor shall be used for joining all outer skin plates.

The beveled side of joints (back face of outer skin plates) shall be welded first, employing "short-arc" or "dip-transfer" or equal type weld procedure, using type 308 welding wire; this weld shall penetrate full depth of bevel. The outer portion of joints shall, then, be welded by Tungsten Inert Gas Process, using small diameter (such as 0.040") electrodes, Argon gas and straight polarity current. Welding details and procedures including proper preparation of plate edges shall be developed for the various welding positions encountered in shop and field assembly. The entire method of joining shall be subject to approval of Contracting Officer, based on test welds made on trial basis.

4-6.6 WELDING: All Welding shall be done in manner to prevent warping and buckling of component parts and assemblies. Members distorted by heat of welding shall be straightened. Surfaces of members to be welded shall be free from loose scale, slag, grease, paint, or other foreign matter; if surfaces within 3 inches of welds have been painted, paint shall be removed prior to welding.

Temporary devices employed to align, clamp or position any plates of outer skin shall be of approved type which will not puncture, deform or deface the work; use of temporary erection bolts for either outer or inner skin plates will not be permitted. All temporary devices employed shall be removed after welds are completed.

Field welding of seams and joints between components and between sub-assemblies shall be equal to and match shop welded work in all respects, and shall produce welds of uniform width and appearance.

FIELD ERECTION

4-7.1 GENERAL: Field erection work shall be in full conformity with contract drawings and procedures established as specified under Section 2: ARCH STRUCTURE - GENERAL. It is emphasized that continuous checking of the rising arch legs against design position (elevations, curvature, etc.) must be undertaken, as outlined in Section 2.

Coordination of arch shell erection with placement of concrete core, as outlined in Section 3: Concrete Work, and with work of other trades, is the full responsibility of Contractor.

4-7.2 FIELD CUTTING AND FITTING: Mismatching of components and pre-assemblies, and errors in fabrication or deformation of components and pre-assemblies, which would prevent proper fitting and assembly of work in field erection shall be reported immediately to Contracting Officer and his approval of method of correction obtained.

4-7.3 FIELD WELDING: All field welding shall be executed under the same qualifications as specified in Articles 4-6.1 through 4-6.6, by specially qualified welders and with the use of adequate equipment in first-class condition.

4-7.4 BOLTING: Standard steel bolt studs, ~~as shown on plans~~, shall be installed during shop sub-assembly of steel skins, to space outer and inner skin properly in relation to each other. Nuts shall be pulled up tight and peened or spot welded to assure fixity of position.

High-tensile bolts (ASTM-A325) shall be installed during shop sub-assembly of steel skins, but nuts shall not be tightened until after concrete core has been placed.

Prior to tensioning of any series or groups of longitudinal post-tensioning bars or strands in concrete core of arch shells, as described in Section 3, STRUCTURAL CONCRETE, all horizontal high tensile strength studs and bolts installed in conjunction with metal skins shall be tightened to not less than the proof load given in ASTM A-325, by calibrated wrenches and in accordance with AISC Specifications for Structural Joints using ASTM A-325 Bolts.

4-7.5 FIELD CLEANING AND FIELD PAINTING:

a. Field Cleaning: Prior to placing fresh concrete against metal surfaces, such metal surfaces shall be cleaned free from grease, oil, loose rust scale and other foreign matter detrimental to concrete; bolts and connectors shall have been treated, as specified in Art. 4-3.4, to break concrete bond.

After erection, surfaces of carbon steel exposed to interior of arch proper shall be (1) cleaned by methods specified for shop cleaning, (2) cleared of all incrustations and splatterings of concrete, including grout and mortar, (3) prepared for field touch-up paint.

Protection on stainless steel shall be removed at proper time or time during progress of construction, as agreed upon in advance between Contractor and Contracting Officer, and stainless steel shall be cleaned to remove dust, dirt, metal particles, splatterings of concrete and other foreign matter, employing such methods and materials as are particularly recommended in writing by manufacturer of stainless steel employed in the work, and approved on trial basis by Contracting Officer. ~~Welds, and weld halos and~~ Weld halos and weld flux shall be removed by means of ~~thods~~ chemical etching, employing methods and agents approved by Contracting Officer.

b. Field Painting: After erection, all carbon steel work shall be cleaned as specified, then, all carbon steel (1) which has not been shop painted (2) from which shop paint has been removed for any reason, (3) field welds, field bolts, nuts, washers, and (4) parts and components within cavities between inner and outer skins of arch shell above Station 41 (concrete core), shall be touched up with same paint used for shop paint. Application of field touch-up shall conform with requirements specified in Article 6-5.9 of Section 6.

Subsequent finish painting is specified in Section 11, PAINTING.

4-7.6 AIR EXHAUSTS: Exhaust air louvers shall be formed in outer skin plates, and corresponding framed openings through inner skin plates formed as indicated and detailed. Outer skin plates shall be cut and louver blades factory-formed integrally with plates in accordance with indicated crosssections.

12. Add a new paragraph 4-7.7, entitled HATCH, page 4-9, reading as follows:

"Provide a 3.4' x 2.0' flush hatch adjacent to Station 0. This hatch is to be constructed of identical materials, as adjacent areas. This hatch to be gasketed and bolted in place and to be removable from the interior. Location and design to be approved by the Contracting Officer.

Appendix B – Finishes Analysis

APPENDIX B – FINISHES ANALYSIS

As part of the HSR, limited analysis to identify the historic paint colors on select painted elements was conducted on the Gateway Arch.

Methodology

Locations of finishes samples extracted for analysis were selected based on findings from exposure craters (approximately one centimeter exposures of the extant layers) conducted while on-site, our understanding of the monument, and areas identified by NPS as being of special interest. Samples were taken from elements likely to represent a variety of different treatments, primarily in public areas.

Small-scale exposure craters were used to identify the extant coatings; given the architecture of the monument; however, exposure windows necessary to identify decorative multilayer finishes systems such as stenciling or imitative finishing and metallic leafs were not conducted as these finishes were not likely used.

All samples were viewed under reflected light microscopy. Initially, the unmounted samples were viewed with a stereomicroscope under 10x to 63x magnification. Selected portions of the samples were prepared for microscopic visual analysis. Preparation of the samples included mounting them in resin prior to grinding and polishing to expose the cross section. The prepared cross-sectioned samples were analyzed with reflected light supplied by a quartz halogen light source equipped with a daylight-balanced filter under magnification ranging from 10x to 200x. The light source used was in compliance with ASTM D1729, *Standard Practice for Visual Appraisal of Colors and Color Differences of Diffusely-Illuminated Opaque Materials*. Additional “destructive” analyses were conducted to supplement the cross sectional analysis by separating the finish layers of the sample during examination.

The earliest exposed coating layers were given a Munsell color number, a scientific alpha-numeric-based system used to describe colors, if possible. The Munsell color number that matched the color closest was chosen, but the color match may not be exact. Color matching was done in accordance with ASTM D1535, *Standard Practice for Specifying Color by the Munsell System*.

In the color grading given in the following table, “dark” is used to describe lower color intensities (black added), and “light” is used to describe higher color intensities (white added). Slight variations in naming were used as necessary to help better describe the actual color. Two layers with the same name, therefore, do not necessarily refer to identical colors but rather to two colors within the same family. Representative photomicrographs are included to provide an understanding of the stratigraphies; however, they cannot be used for color-matching because of color shifts that may occur when photographs are taken and printed. The colors illustrated in the photomicrographs generally appear lighter than the actual colors.

On September 1 and 2, 2009, color measurements of the exposed coatings of select elements was measured using a spectrophotometer. The illumination used was D₆₅. Measurements are reported as CIE L*, a*, b* values.

Coating Thickness

At selected areas, the number of coating layers and total dry film thickness were measured in accordance with the applicable provisions of Test Method-A outlined in ASTM D4138, *Standard Test Methods for Measurement of Dry Film Thickness of Protective Coating Systems by Destructive Means*. In Test Method-A, a groove is cut into the film with a carbide tipped wedge at a precise angle. The thickness at the cut is measured using a microscope with a reticle and scale. During this investigation, a Tooke Inspection Gage was employed with a 2x cutting tip; therefore, the dry film thickness (DFT) of the coating could be measured to the nearest 0.5 mil.

Analyses

Table 1 summarizes the samples of applied coatings analyzed, and Table 2 summarizes the results of the color analysis.

Table 1. Paint Sample Locations

Sample	Description
1	North leg, loading lobby, tram door surround
2	Observation deck, ceiling (27mils DFT)
3	Observation deck, closet, ceiling (6mils DFT)
4	Observation deck, ceiling (10 mils DFT). Note: the ribs and plates had the same coatings chronology.
5	Observation deck, tram loading, wall panels at top of loading area (11 mils DFT)
6	Observation desk, tram loading area, railing support, (11 mils DFT)
7	Observation deck, stair stringer
8	North leg, interior skin
9	North leg, interior skin
10	Attic stock, tram door, lobby side
11	Attic stock, tram door, tram side
12	Tram capsule, exterior, (5 to 7 mils DFT)
13	Tram capsule, interior walls (13 mils DFT)

Table 2. Analysis Summary Table

Sample	Primer	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Layer 7
1	Red	Gray green (unexposed intermediate coat) 5GY 6/2	Gray 10PB 7/1	Gray	Off-white	White	Gray	Metallc gray
2	Red	Gray green (unexposed intermediate coat) 5GY 6/2	Gray 10PB 7/1	Gray	Gray	Off-white	Off-white	Off-white
3	Red	Gray green (unexposed intermediate coat) 5GY 6/2	Gray (note top surface has yellowed)					
4	Red primer	Yellow gray/green (intermediate coat)	Gray 10PB 6/1	Off-white				
5	Yellow gray/green	Gray N4.75	Gray	Off-white	Off-white	Metallic gray		
6*	Red primer	Gray	Off-white	Off-white	Gray			
7	Red primer	Yellow gray/green (intermediate coat)	Gray 10PB 6/1	Off-white	Off-white	black		
8*	Red primer	Gray (darkened on surface)						
9*	Red primer	Gray (darkened on surface)						
10	Off-white (primer)	Gray						
11	Yellow gray/green (primer)	Light blue						

Sample	Primer	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Layer 7
12	Yellow green	Yellow green						
13	Light blue	Light blue	Light blue	Light blue	Light blue			

* indicates study limited to field analysis

Discussion

No testing for hazardous materials such as lead has been identified. All paints are presumed to be lead containing.

Observation Deck

The ceiling of the observation deck consists of white painted steel plates and steel ribs (Figure B-1). Samples 2, 3, and 4 represent the ceiling and likely contain complete stratigraphies. The ceiling was originally primed with a red primer and a gray green intermediate coat. The original exposed finish of the ceiling is gray in color, closely matching Munsell colors 10PB 7/1 and 10PB 6/1 (Figure B-2). The original color remains exposed in the ceiling of the closet at the observation deck (Figure B-3). The color of the closet walls at the observation deck at the north was measured using a spectrophotometer. The average color of the closet at the observation deck at the north was $L^*=51.92$, $a^*=-0.36$, $b^*=4.66$. The color of the closet at the observation deck at the south was measured using a spectrophotometer. The average color of the closet at the observation deck at the south was $L^*=63.99$, $a^*=-0.50$, $b^*=7.11$. The ceiling subsequently has been painted a series of grays and off-whites. The walls to the observation deck are currently carpeted. The current carpet was reportedly installed in approximately 2002.

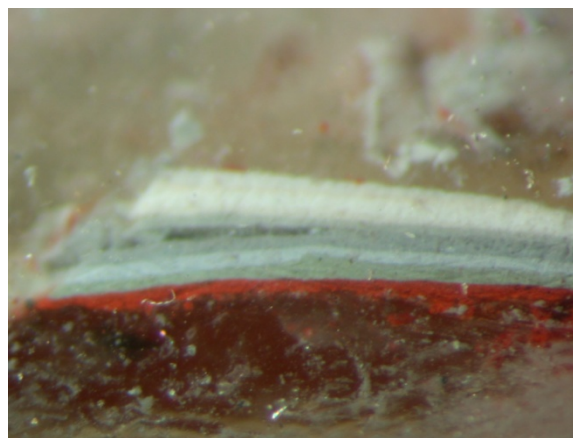


Figure B-1, left. Overall view of ceiling at observation deck. Source: WJE, 2008. Figure B-2, right. Photomicrograph of Sample 2, removed from the ceiling of the observation deck. Red layer indicates primer coat. Source: WJE, 2009.



Figure B-3, left. Overall view of ceiling at observation deck closet. The gray color likely represents the original ceiling color in the observation deck. Source: WJE, 2008. Figure B-4, right. Overall view of observation deck tram loading area. Source: WJE, 2008.

Observation Deck Loading Lobby

The white painted steel ceiling panels and the carpeted walls in the observation deck continue into the tram loading area (Figure B-4). The walls on the tram entrance are currently painted a metallic gray color (Sample 5). The samples likely contain the original coatings. The wall panels were originally painted a gray color closely matching Munsell N4.75 (Figure B-5). The railing supports and stair stringers were also originally painted gray, closely matching Munsell color 10PB 6/1.

It is likely that all the gray colors observed as the earliest finish at both the observation deck and the observation deck loading lobby were intended to be the same color gray. Variations in color are likely a result of different exposures or slight variations in original coatings.



Figure B-5. Photomicrograph of coating sample removed from wall at tram entrances. Source: WJE, 2009.

Trams Capsules

The tram capsule doors were reportedly stripped of existing coatings and repainted previously. Samples from attic stock of tram capsule doors stored in the north leg were analyzed (Samples 10

and 11). The color of the tram capsule doors was measured using a spectrophotometer. The average color of the attic stock tram capsule door was $L^*=88.38$, $a^*=-5.45$, $b^*=-3.49$.

The exterior of the tram capsules (Sample 12) and the interior of the tram capsules were analyzed (Sample 13). Based on the analysis, the original coating does not seem to be present on the interior or exterior of the trams; therefore, Munsell color matching was not completed. The green color of the exterior of the trams was measured using a spectrophotometer. The average color of the exterior of the tram capsule was $L^*=45.74$, $a^*=-7.79$, $b^*=20.76$. The original color of the exterior of the capsules is unknown. The color of the paint at the interior of the capsules matches the interior of the capsule doors. All of the coatings were reportedly removed from the tram cars previously.

The original paint is present and exposed on the attic stock doors, although it appears to have slightly darkened based on the appearance of the coating just below the exposed surface. The inside of the doors had two coating layers, both light blue paint with a total thickness of 5 mils DFT. The color was measured using a spectrophotometer. The average color of the attic stock tram capsule door was $L^*=76.64$, $a^*=-5.37$, $b^*=-8.69$. The color difference between the original attic stock doors at the existing coating on the tram doors is 12.83 ΔE . While both colors are similar light blues, based on previous experience a ΔE of greater than 5 is generally considered significantly different.

Two attic stock doors from the trams (Figure B-6) were analyzed. The doors were reportedly never installed. The original paint is present and exposed on the attic stock. The lobby side of the door had two coating layers. There is a thin light off-white primer (0.5 to 1.0 mils DFT) and a gray top coat with a thickness of 3.5 to 4.0 mils DFT.

A letter from Bruce R. Detmers of Eero Saarinen and Associates to the MacDonald Construction Company dated April 7, 1964, indicated that the tram capsules were to have a “vinyl plastic finish” and that the interior of the walls, doors, and lights was to be “blue No. E-57.” While the specific paint specified for the capsules is not identified, generically vinyl coatings “evolved as the first really premium coatings as a result of performance during and after World War II.”*

* Guy E. Weismantel, *Paint Handbook*. (New York: McGraw-Hill Book Company, 1981), 14–32.



Figure B-6. Overall view of attic stock tram doors with original blue coating on the interior of trams exposed.
Source: WJE, 2008.

Tram Loading Area

In the north tram lobby, the exposed surface of the tram doors is brushed white metal with a clear coat. The surrounds are currently painted with a metallic gray coating (Sample 1). The exposed color of the load zone lobby side tram door surrounds was measured using a spectrophotometer. The average color of the metallic gray of the north leg doors was $L^*=70.02$, $a^*=-1.15$, $b^*=0.43$.

In the south tram lobby, the exposed surfaces of the tram doors are painted with a gray coating. The exposed color of the load zone lobby side tram doors was measured using a spectrophotometer. The average color of the gray of the south leg doors was $L^*=62.22$, $a^*=1.01$, $b^*=5.24$.

The color of an attic stock load zone door in the north leg was measured using a spectrophotometer. The average gray color of the load zone doors was measured to be $L^*=59.93$, $a^*=-0.96$, $b^*=1.84$.

Sample 1 appears to contain the original coatings. Based on the analysis, the original coating likely was gray in color, closely matching Munsell color 10PB 7/1. The color difference between the original attic stock lobby doors and the existing coating on the doors is $4.55 \Delta E$. Based on previous experience a ΔE of less than 5 is generally considered matching.

Interior Skin of Arch Legs

A limited condition assessment of the coating on the interior surfaces of the legs of the arch was conducted, including Samples 8 and 9. A complete historic finishes analysis was not completed since this area represents a non-public space. Within the north leg of the monument two coating layers were identified: a red primer approximately 3 to 7 mils DFT and a gray top coat 3 to 4 mils DFT thick. At isolated areas, un-feathered edges were observed, suggesting touch-up repairs to the coating likely subsequent to original construction (Figure B-7). Isolated failures of the coating system were identified primarily around bolts where there was evidence of water leakage or condensation (Figure B-8). At some areas a white deposit was observed, likely efflorescence from the concrete fill on the coating (Figure B-9). The white deposit effervesced freely when tested with dilute hydrochloric acid, suggesting that the deposit was calcium carbonate. The calcium carbonate is likely a result of water dissolving calcium hydroxide in the concrete between the interior and exterior skin and dripping through the bolt holes.

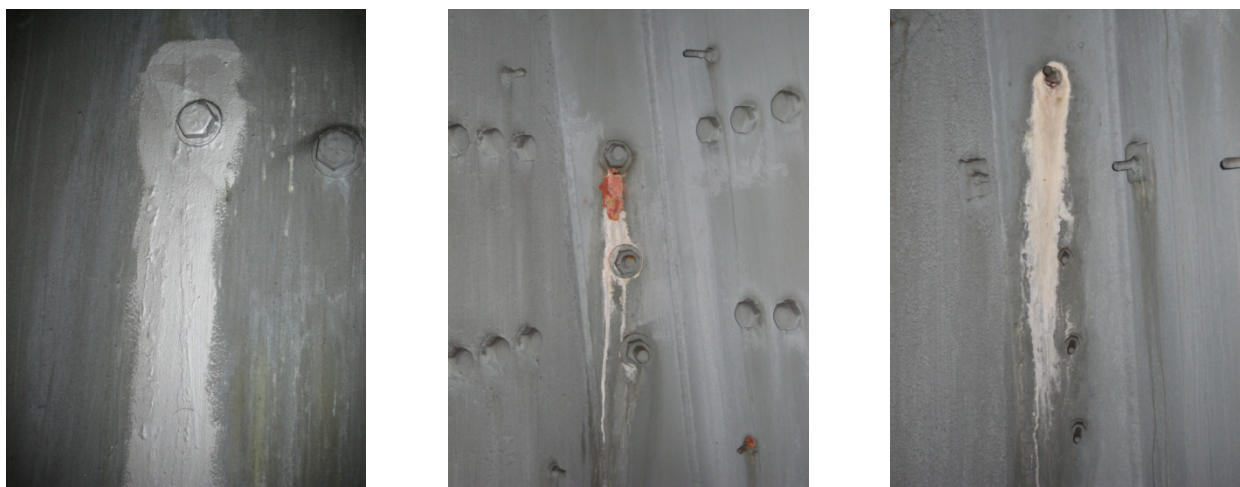


Figure B-7, left. Area of touch-up paint in inner skin in north leg. Note that edges of the adjacent coating were not feathered smooth. Source: WJE, 2008. Figure B-8, center. Area of paint failure exposing primer and corrosion and white deposit (calcium carbonate) below bolt hole at inner skin of north leg. Source: WJE, 2008. Figure B-9, right. Area of calcium carbonate build-up below bolt hole at inner skin of north leg. Source: WJE, 2008.

The revised specifications entitled “Painting Instructions” dated September 11, 1962, identify that the carbon steel was to be solvent cleaned (SSPC-SP1) and hand cleaned (SSPC-SP2), and coated with “one (1) coat of shop primer per SSPC-Paint 2-55T, ‘Red Lead, Iron Oxide, Raw Linseed Oil and Alkyd Primer.’ Two (2) coats of primer shall be applied to areas which will be unaccessable [*sic*] after erection.”[†] It is unclear from the specifications reviewed whether the gray top coat was considered the second coat of primer.

The color of an exposed gray coating in the north leg was measured using a spectrophotometer. The average gray color was measured to be $L^*=51.47$, $a^*=-1.76$, $b^*=-1.61$.

[†] Pittsburgh-Des Moines Steel Company, *Painting Specification*; Drawing P1 (Sheet 1 of 2), dated 8-2-62, Rev 9-11-62.

Recommendations

- Restore historic finish colors in accordance with finishes analysis. Note period of significance in selecting colors for restoration.
- Conduct analysis of hazardous materials RCRA-8. The designation RCRA-8 refers to the materials identified in the Resource Conservation and Recovery Act and includes arsenic, barium, cadmium, chromium, lead, mercury, selenium, and silver.
- Touch-up paint within legs where isolated paint failure has occurred, primarily at bolts.

Appendix C – Previous Structural Studies

- Rose, Edwin, and Harvey C. Olander. *Report on the Jefferson National Expansion Memorial Gateway Arch*. Denver, Colorado: U.S. Department of Interior, Bureau of Reclamation, Division of Design, Office of Chief Engineer, December 1964. Not reproduced herein are the appendices of this report, which include Appendix A: Discussion and Determination of Wind Forces; Appendix B: Thermal Studies; Appendix C: Earthquake Forces; Appendix D: Laboratory Tests; Appendix E: Structural Model Tests; and Appendix F – Volume I: Computer Analyses. (157 pages)
- Cayes, Lloyd R., and Charles F. Scheffey. Memorandum from the Bureau of Reclamation, excerpts of conclusion from *Aerodynamic Stability of the Jefferson National Expansion Memorial Gateway Arch*. Denver, Colorado: Department of the Interior, July 1965. Including Appendix A: Field Study of Partially Completed Arch, April 1965.1 (46 pages)

32.1

United States
Department of the Interior
Bureau of Reclamation

Report on the

JEFFERSON NATIONAL EXPANSION MEMORIAL GATEWAY ARCH

by

Edwin Rose

Chief, Structural and Architectural Branch

Harvey C. Olander

Head, Bridges Section



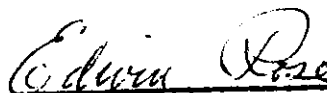
**Division of Design
Office of Chief Engineer
Denver, Colorado**

December 1964

FOREWORD

This is our report on the analysis of the design of the Jefferson National Expansion Memorial Gateway Arch, prepared at the request of the National Park Service. The report presents our findings on the structural adequacy of the arch and recommendations for changes in the design. In arriving at our recommendations, we were aided by the full technical facilities and resources of the Office of Chief Engineer of the Bureau of Reclamation at Denver, Colorado.

This volume of the report contains five appendixes. The sixth appendix on computer analysis of the arch is contained in a second volume, issued separately as Appendix F.



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Office of Chief Engineer
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Denver, Colorado

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*This appendix is being issued as a separate volume.

Chapter I

INTRODUCTION

The Jefferson National Expansion Memorial Gateway Arch, a 630-foot-high, stainless steel arch symbolizing "St. Louis - Gateway to the West" will be the central feature of the Memorial commemorating the Louisiana Purchase and the Nation's westward expansion. The Memorial, now under construction at St. Louis, Missouri, covering an area of about 40 city blocks, includes an underground visitors' center at the base of the arch and is under the administration of the National Park Service.

The hollow triangular core of each leg of the arch will accommodate transportation facilities from the visitors' center to an observation platform at the top of the arch. The principal transportation housed in each leg will be a 40-passenger train consisting of eight 5-passenger cars capable of operating to within a few feet of the top. Supplementing the trains are elevators in each leg rising to a height of 370 feet. Each leg also houses a stairway to the observation platform.

On March 14, 1962, a contract was awarded the MacDonald Construction Company for construction of the Gateway Arch and Visitors' Center. The prime subcontractor to the general contractor is the Pittsburgh-Des Moines Steel Company, fabricators and erectors of the steel of the arch. The construction of the arch proper appears to have started in the spring of 1963.

By the letter of February 10, 1964, from the Acting Commissioner of the Bureau of Reclamation, the Bureau's Chief Engineer was requested to provide technical assistance to the National Park Service in resolving the complex problem regarding the structural design of the Gateway Arch. An independent review and check of the structural design of the arch was required. Since construction of the arch was above the 160-foot level as of January 1964 and fabrication completed for sections for a considerably higher level, we urgently pursued our studies and submitted an interim progress report June 11, 1964, to present our findings and recommendations as soon as possible.

To accomplish this assignment, we requested specifications, design data and assumptions, computations, test data and special studies, and updated construction drawings pertinent to structural design. The results presented herein are based on the data provided and such additional information made available to us during the investigation.

Review of the available material raised questions regarding some of the data and assumptions, particularly regarding the aerodynamic stability of the arch. To investigate aerodynamic stability, we

decided in March 1964 that wind tunnel tests were required on a model of the arch scaled to similitude requirements for stiffness, mass, and geometric relationship of the final design of prototype. The results of the aerodynamic investigations are described in a separate study and are not included herein.

Described briefly, the arch has a basic shape of an inverted catenary curve with the arch legs equilateral triangular in cross section. Both the height and span are 630 feet. The basic structure of the arch comprises a double-skin wall with the space between the inner and outer steel skins varying from 36 inches at the base to 7-5/8 inches at a height of 390 feet and above. The outer skin is 1/4-inch stainless steel throughout. The inner skin is 3/8-inch type A-7 carbon steel except at the corners where it is 1-3/4 inches thick. Up to a height of 300 feet the space between the skins is filled with concrete and the concrete core post tensioned. Above the 300-foot level, the two skins are connected by carbon steel diaphragms running longitudinally at 2-foot centers. This arrangement forms a welded cellular-type structure similar to certain aircraft structures. The base of each leg is anchored by the post tension bars and anchor bolts into an independent foundation of mass concrete which is founded on rock.

Drawing No. XOA-D-909 gives a sketch of the arch and simplified typical sections and an isometric view showing some of the principal structural elements.

The arch geometry and detailed structural dimensions are shown on referenced drawings. 2/, 3/*

*Numbers indicate references in list at end of report.

Chapter II

SUMMARY AND RECOMMENDATIONS

1. Summary

(a) General

This report presents results of an independent review and analysis of the structural design of the arch based on drawings and data available to us from the National Park Service, their consultant architectural designer, Eero Saarinen and Associates, and the structural engineering consultant designers, Severud-Elstad-Krueger Associates. This report does not include the results of studies of aerodynamic stability of the arch, which are the subject of separate investigations. Aerodynamic stability cannot be assured until successful completion of wind tunnel tests.

The arch is of unique structural design, particularly in its shape and the variety of structural materials and elements assembled into a very large monolithic structure. For design of such a structure, special studies and tests are necessary to establish design criteria and verify methods of design. Reliance cannot be placed on building codes, which establish minimum requirements only for conventional structures.

Our design criteria were established by review and extensive supporting studies of historical records and probability studies of climatological data. In addition, we carried out special studies to determine magnitude of wind loads, earthquake loads, and thermal gradients through the arch wall. We also conducted tests of structural models of critical segments of the arch and material tests.

Because the arch was constructed to a height greater than 160 feet before the Bureau study was requested, our approach to the problem was different and our procedure was necessarily more restricted than if the Bureau had been consulted before construction was initiated. Our procedure would have been to conduct wind tunnel tests on a model of the final design of the arch scaled to similitude for mass and stiffness as well as geometry to assure aerodynamic stability. We also would have considered it essential to conduct extensive research into physical and thermal properties of all structural materials and to conduct structural model research, in addition to thorough analytical design, before proceeding with construction of such a unique monumental structure. Finally, a program of instrumentation would have been undertaken to measure temperatures and strains at certain critical points in the arch during the construction and after completion of the arch.

An interim progress report was submitted June 11, 1964, pointing out inadequacies of certain structural steel details in the arch above Station 45 and making recommendations for their correction. After the June 11 report was submitted, a meeting was held June 22 with representatives of the National Park Service, Architect Saarinen and Associates, and Engineering Consultant Severud. The consultants presented additional information regarding the latest design changes, including a shorter gap in longitudinal diaphragms and addition of an interior strut at each of the transverse triangular frames. The National Park Service and the consultants requested that we recheck the design, including these modifications, and also give further study to metal temperatures.

Since the June 22 meeting we have made extensive structural analysis of a typical transverse frame at arch Station 15. The rigid corner detail and its highly indeterminate stiffness and stress distribution constitute a major problem. Structural tests were performed on models of a typical rigid corner of the triangular frame and also on a model of the cellular wall of the arch rib with the discontinuous longitudinal diaphragms. Heat transfer studies and experimental studies were also made to support our temperature design assumptions. The basic arch and the typical transverse triangular frames were analyzed for a wide range of loading combinations of temperature and wind conditions established from historical records, probability studies, and heat transfer analyses.

In recognition of the advanced stage of construction and the need to keep design changes to a minimum, we made extensive studies and tests to try to salvage the original design (with rigid corners). It appears possible to develop a modified design of the rigid corner of the typical transverse triangular frame selected at Station 15 which will have a minimum factor of safety. However, analysis of the transverse frame at Station 44 shows that a more radical change will be required in this area. If the cellular type of design is retained, each section above Station 45 must be checked in the same detailed manner as was done for Station 15. Sample calculations for Station 15 are included in Chapter V as a guide for such checking.

At the invitation of the Bureau of Reclamation, another meeting was held on September 22, 1964, with the National Park Service and the architectural and structural engineering consultant designers to discuss the results of our studies since June and to advise them in advance of the principal findings we plan to present in our report and to give them opportunity to present any design changes they have made. No changes in design or additional supporting data for the present design were presented by the consultants. Their principal comments were in criticism of our criteria as being too severe.

At this point it appears appropriate to discuss the design data, calculations, studies, and tests made available to us by the National Park Service, the Architectural Consultant Saarinen, and the Structural Engineering Consultant Severud. When we were first assigned this work, we met in the New York office of Severud Associates, and it was agreed all the latest data and studies would be sent to us promptly. We did receive shortly after this meeting contract drawings, specifications, change order data, and the following calculations, studies, and tests, which to our knowledge are all that were made previous to our September 22 meeting.

- Severud-Elstad-Krueger Associates Data Book 3220 (subsequent computations received March and April 1964)

- Report by D. B. Steinman, Consulting Engineer, New York, entitled "Jefferson Memorial Arch--Aerodynamic Studies," December 31, 1948

- Report by D. B. Steinman entitled "Jefferson Memorial Arch--Supplementary Aerodynamic Studies," February 24, 1949 (photographs missing)

- Report No. 314-001 by Fairchild Aircraft and Missiles Division, entitled "Jefferson National Expansion Memorial Arch--Preliminary Dynamic Analysis," April 27, 1960

- Report No. R314-002 by Fairchild Stratos Corporation, entitled "Jefferson National Expansion Memorial Arch--Dynamic Analysis," October 23, 1961

- Undated report in German, entitled "Sicherheitsnachweis für seit liches Ausknicken des Bogens" (translation: "Safety Analysis of the Arch Against Sidewise Buckling") by Dr. Konrad Sattler, Technical University, Berlin

In the work described above, it appears no studies were made regarding specific weaknesses we believe are present in the structure. We also believe that the forces contributing to the weaknesses were not sufficiently considered. These are discussed point by point as follows:

(a-1) No tests or studies were made regarding the effects of stopping the longitudinal diaphragms between the exterior and interior skin plates and thus creating a gap in the diaphragms.

In our meetings, Structural Engineering Consultant Severud and his associate Dr. Bandell made reference to specifications that permit longitudinal stiffeners to be stopped short, thus creating discontinuities. Upon our pressing for clarification on this point, they then referred specifically to the American

Association of State Highway Officials' "Standard Specifications for Highway Bridges" and the paragraph on use of longitudinal stiffeners to stiffen webs of girders in bending, Paragraph 1.6.81.

The design of plate girder stiffeners is an entirely different and a much less complex problem than we are considering in the sides of the arch. First, the location of longitudinal stiffeners in plate girders is in a region of low stress; therefore, it follows the stress concentration they create in the web is not critical. Second, the stress concentration factor in plate girders is small compared to that created by the longitudinal diaphragms of the sides of the arch. Whereas the concentration factor for the arch is 3.3, the factor for the plate girder is approximately 1.75, assuming an ordinary stiffener angle attached to one side of the web plate. Third, the total resisting moment of the girder comes largely from the girder flanges, the web contributing little. Failure of this portion of the web would not produce a crippling effect on a plate girder that is at all comparable to the seriousness of separating the inner and outer skin plates of the arch wall. This separation will be the result of high stress concentrations at the ends of the diaphragms if they are not made continuous.

We therefore feel that the provisions of Paragraph 1.6.81 of American Association of State Highway Officials' "Standard Specifications for Highway Bridges" cannot be accepted as applicable to the arch problem. Specific studies and tests are needed to determine the effect of discontinuities in the diaphragms. Although our studies are of necessity limited, the two tests referred to in Chapter V of this report substantiate our contention regarding this point.

(a-2) No tests or studies were made to determine a rational basis for analysis of the rigid-type corners of the transverse triangular frames. The type of welds called for in the design drawings definitely points out that one of the critical sections, the one that bisects the corner, was not properly analyzed. There are many studies of rigid corners recorded in available literature, see references 13/, 14/, and 15/, that are pertinent to the problem. The acute 60° angle creates an even more critical section than the 90° corners used in the reference studies. This matter is also discussed in Chapter V.

(a-3) Studies regarding aerostatic wind forces were not made. The data which were used are, in general, not applicable. The specifications that were used in the design, "Minimum Design Loads in Buildings and Other Structures" by American Standards Association, September 3, 1955, state under Chapter 5--Wind

Loads, Section 5.1, "Minimum Design Pressures," that " * * * design pressures were computed applying a shape factor 1.3 (the effect of pressures on exterior surfaces of ordinary rectangular buildings)." The arch does not approximate the shape of a rectangular building. Under "Other Structures" it is also stated, "The building official may require evidence to support the values for wind pressure used in the design of structures not specifically covered in this section." Also, "Design pressures on signs located 500 feet or more above ground should be determined by special analysis of conditions."

(a-4) Thermal studies made only touched upon the true nature of the problems involved. Because there are little data available on the subject, research is needed to determine thermal effects. Limited tests were performed by the Bureau, see Appendix B. ✓

(a-5) The physical and thermal properties of stainless steel should have been thoroughly investigated. It is our opinion that if this had been done the advantages of using stainless clad plate on A-7 carbon steel base would have been recognized.

(a-6) The stability of the arch as a whole was not thoroughly checked, in spite of strong recommendations made in Mr. D. B. Steinman's report of February 24, 1949. A partial analytical study was made in the undated report (in German) by Dr. K. Sattler of Technical University, Berlin, but this was very incomplete; in fact, the report appears to have been intended to serve only as a guide for making the specific studies for the arch. This study went only so far as to consider the buckling of the arch as a whole but only under its own dead load. The study did not take into account stability when aerodynamic forces are considered.

There is also the study made by Fairchild Stratos Corporation which appears to prove that the problem of vortex shedding is not inherent in the arch because of the varying cross section. Here again, this does not give a complete answer because there are factors other than vortex shedding to bring about instability. There is an example in actual practice that does in fact cast doubt on the conclusions arrived at by this report. We refer to the double intersecting arch supporting the restaurant at the Los Angeles Airport which is discussed in a paper entitled "Aeroelastic Vibration of a Steel Arch" in the June 1964 issue of the Journal of the Structural Division, American Society of Civil Engineers. The stability of structures under aerodynamic forces is a very complicated matter; and in probably all the cases where it has been critical, the problem has been studied by use of models in wind tunnels.

Although this was recommended in both the Steinman reports, the recommendation was not carried out in the final design of the arch.

(a-7) Although our studies are comparatively comprehensive, many more points at different stations must be investigated, particularly in the region of the crown of the arch, if the present design is carried out to completion.

(b) Basic Design Approach

We have approached the problem from the generally accepted standpoint of keeping stresses within the elastic limit and permitting no buckling. In the aforementioned conferences with the architectural consultant and engineering consultant, they suggested that this approach is too severe and has been outmoded by the concept of plastic design and ultimate strength beyond buckling. A commentary on plastic design in steel 30/ states, regarding its limitations, "In ordinary building construction, limitations such as fatigue and buckling are usually the exception and not the rule. Therefore plastic design is finding considerable application in continuous beams and frames where members are stressed primarily in bending." The structural steel portion of the arch as designed is definitely subject to reversal of loads and buckling conditions. Neither is it primarily limited to bending stresses. Furthermore, the analysis would have to be carried out quite extensively to determine how yielding of any part affects the overall stability. This type of analysis is possible for relatively simple structures where only a few members are involved and usually where there are only static forces to consider. However, in an extremely complicated structure such as the arch, subjected to innumerable combinations of loads created by wind from any direction, varying temperatures, and the fact that the nature of the wind forces themselves are very capricious, a meaningful plastic design is virtually impossible.

There has been a new approach in recent years regarding buckling; and, in effect, buckling is now permitted in some specifications. However, the specifications provisions are limited primarily to girder and beam webs and isolated members. Buckling was permitted only after extensive research was carried out specifically for such limited uses. Here again, we consider the arch to be much more complicated than simple beam or girder webs or isolated compression members because buckling of local portions of the arch may affect the overall properties of the structure.

To investigate a design considering the effects of local buckled areas which may shift with every change of wind and variation of temperature, would increase engineering costs. It is more economical to invest in heavier materials. It is also doubtful that

wind tunnel tests, to be completed later, could possibly include the effects of buckled plates.

(c) Welded Cellular Construction

The arch is designed as an all-welded structure. The cross section of the rib is an equilateral triangle whose sides are composed of two skin plates separated by diaphragms which also make the two skin plates act together. This may be called a cellular-type of construction and is very similar to that commonly used in certain aircraft structures. Neither plate is stable by itself; one depends on the other for stability. Therefore, effective and reliable connections of diaphragms to skin plates are absolutely essential to assure that the cellular arch rib functions permanently as a structural load-carrying unit. It should also be noted that where welded construction is used in cellular aircraft structures an annealing or heat treatment procedure is used to reduce residual stresses. Unknown stresses due to rolling the plates and due to welding will exist in the arch plates. In the case of aircraft structures, very extensive tests on a full scale model are conducted before approval of the design.

In the arch the diaphragms are connected to the outside stainless steel skin plates by fillet spot welds. Test results were reported by the consultants that these welds develop ultimate strength in excess of 7,000 pounds per weld 21/. However, they are made by a new welding procedure and are untried in actual practice. In standard practice, spot welds are not permitted for stress carrying purposes. Although individual welds may be tested for static loads, this does not tell us how the welds will behave under actual conditions of repeated fluctuating loads and locked-up stresses due to welding.

In the arch the standard fillet weld originally specified is not to be used because it distorts the outside skin plate and causes plate warping, thereby creating an unsightly appearance. When the sections of the arch are completed, the spot welds are completely hidden from view, and it is impossible to examine them thereafter. If failures occur in these welds, they will not be evident until some form of critical failure has occurred. The same is true of the corners of the frames if they are made rigid. Although we made the best evaluation possible as to effective plate widths as explained in Chapter V, and have calculated corner stresses on the basis of these assumptions, this is an area of speculation.

Here then are two critical points; namely, the "double spot fillet welds" and the rigid corner of the transverse frame that will

always be in question with no possible way of examining them except by some hit-or-miss spot-check method which will be difficult and expensive to perform. This is undesirable, and our judgment is that safety factors of the magnitude of three or more should be used for normal loading conditions in such cases. This would rule out the present design almost entirely; otherwise, we are forced to ignore our judgments based on past experience and accept low factors of safety, and there is no engineering justification for doing this.

(d) Inadequacies in Present Design above Station 45

It is our considered opinion that there are inadequacies in the structural steel design details above Station 45 as follows:

(d-1) The discontinuity in the longitudinal diaphragms, see Drawing No. X-OA-D-909. This gap causes a high stress concentration in the skin plates in the region near the ends of the diaphragms.

(d-2) The rigid corner detail of the transverse triangular frames. This rigid corner causes very severe local stresses which create elastic instability conditions. Transverse thermal stresses caused by rigid corner details contributes largely to these.

(d-3) Because of the high thermal coefficient of expansion of stainless steel plate, temperature stresses are very high. This will necessitate a change in the vicinity of Station 44 and above of the 1/4-inch stainless steel plate to stainless clad steel plate. The extent of this change above Station 44 has not been determined.

In addition, the following details are considered undesirable or of questionable adequacy:

(d-4) The use of a new procedure, untried in practice, of "double spot fillet welds" in lieu of continuous fillet welds originally specified for welding the stainless steel exterior plate to the longitudinal diaphragms. Even if these diaphragms are made continuous, this a most critical area in obtaining the effective functioning of the cellular arch rib or wall as a unified structural element. Tests by the Bureau (see Appendix E) indicate that this condition is particularly severe for the present design at those welds at the discontinuous ends of the longitudinal diaphragms. Although test data provided by structural designers Severud et al. on these welds indicate satisfactory strength for the upper part of the arch, close

control of welding and qualification of all welds is essential to assure full effectiveness of all welds.

(e) Inadequacies in Present Design from Station 45 to 71, Inclusive

Below Station 45, stud welds and "double spot fillet welds" for connecting the stainless steel plates to stiffeners to develop the composite section are severely overstressed and will shear or separate from the concrete due to creep, shrinkage, and temperature change. This will render the stainless steel exterior plate ineffective as a part of the composite section and may create undesirable effects on the appearance of the arch. Our preliminary investigations indicate that other design deficiencies exist in the arch below Station 45, but these may not be critical.

As the lower portion of the arch has been constructed, it is now extremely difficult to correct these inadequacies or to make any changes. Strengthening of this part of the arch can only be done by adding structural members to the arch. Although the lower portion of the arch was not completely investigated, a stress study, discussed in Chapter IV, was made for a section below Station 45, assuming the exterior plate not effective and the concrete cracked. This study showed stresses on the interior steel plate are high but not excessive.

We wish to emphasize that we concentrated our studies on the upper portion of the arch because only a small amount of construction on this portion had been completed. In addition, our preliminary investigations indicated design deficiencies in this portion of the arch, and we considered that rectification of these deficiencies were more pressing and required immediate attention.

2. Recommendations


(a) Preferred Recommendations

Since the above inadequacies require drastic changes and arch sections above Station 45 that have already been fabricated must be disassembled and refabricated to change the outside skin plate, we strongly recommend a radical change in design, that is, to replace the present cellular-type construction with a design similar to that shown on Drawing No. X-OA-D-913. This latter design has many advantages. First, the thermal stresses are practically eliminated, whereas the present design is governed by these stresses. Second, the problems of welding as discussed under Welded Cellular Construction are reduced to those normally encountered, and standard methods of welding may be employed. The proposed design makes possible ready access to all parts of the structure and it remains open for inspection.

Our preliminary studies show that this design revision can be accomplished with little or no increase in the amount of material required. The cost of fabrication and erection, however, should be greatly reduced because the design is simplified and lends itself to standard automatic welding of all the parts.

No effort has been spared in our studies to modify the present design so that we could recommend it. However, with all considered improvements the resulting structure will not meet the standards we require for Bureau of Reclamation structures of similar importance. Furthermore, if these improvements are followed, they are going to be very costly and in our opinion will more than offset the cost of abandoning some of the work that has already been done above Station 45 and starting with the new design. It is our opinion that if comparative bids were taken at the outset on the two types of design, the proposed design would certainly be far cheaper than the present cellular design. Therefore, we believe that a vastly improved design with resulting lower stresses can be achieved at little or no increased cost over improving the present design with its uncertainties and high stresses.

The basic design shown on Drawing No. X-OA-D-913 utilizes a stainless clad steel plate as a major structural element. In consideration of uniformity in appearance of the stainless clad steel adjacent to stainless steel plate of the lower part of the arch, an alternative design is also shown on the drawing. This provides a basic structural unit of carbon steel plate with a light gage stainless steel plate separately attached for architectural purposes only.

In view of the high tensile stresses indicated in the concrete in the foundation structure as shown by the stress analysis in Chapter VII, it is recommended the concrete be regularly inspected for structural cracking, particularly in the area on the top of the foundation directly below the arch intrados. 

(b) Alternative Recommendations

If the above suggested change is not followed, the present design can be improved so that computed safety factors are at least greater than unity. To accomplish this, the following changes are essential:

- (b-1) That the longitudinal diaphragms be made fully continuous. Suggested details to permit welding and assembly are shown on Drawing No. X-OA-D-904.

(b-2) That the corner detail of the transverse triangular frame be changed to provide a positive hinged connection as shown on Drawings No. X-OA-D-903 and X-OA-D-905.

(b-3) From Stations 45 through 27 the distances between skin plates be increased as recommended in Chapter V.

(b-4) At Station 45 provision be made for a positive stress transfer from the exterior steel skin plates of the upper portion of arch above Station 45 to the concrete below this station. Drawing No. X-OA-D-906 shows a suggested method of accomplishing this.

(b-5) Outside skin plates in the vicinity of Station 44 and above be changed from stainless steel to stainless clad steel, and certain additional transverse stiffeners be added to these plates as called for on Drawing No. X-OA-D-908. The extent of this change above Station 44 has not been determined.

The above changes, except (b-4) and (b-5), were specifically noted and the same recommendations were made in our second interim progress report dated June 11, 1964. Item (b-4) was mentioned in supplementary comments dated August 10, 1964.

Chapter III

SAFETY FACTORS

It is pointed out in Chapter V that the present design is considered unsatisfactory, with safety factors less than 1.0. Therefore, in the following discussion, the present design is not considered. Rather, two designs that incorporate modifications of the present one are considered.

There are many things to consider when establishing a safety factor, but first of all we must define it. Since safety factor is the ratio of critical stress to computed stress, it follows we must define critical stress.

We stated in Chapter II that our design approach is based on yield point of the material or buckling, whichever applies to the particular case, and gave our reasons for so doing. This approach then defines critical stress as we see it. To repeat, critical stress equals yield point stress or buckling stress, whichever governs. In the case of biaxial stresses, critical stress is defined by the Maximum Distortion Energy theory of failure, or buckling due to stresses in two directions.

Consideration should then be given to the nature, magnitude, and frequency of the loads assumed. In previous discussions of wind loads, we recommended that studies of the arch be based on the 50-year wind, which gives a basic wind velocity of 75 miles per hour. The same data 5/ from which this was obtained would give a velocity of 78 miles per hour for a 100-year period. Recent probability studies 16/, based on a 47-year period of record at St. Louis, give basic wind velocity of 71 miles per hour for the 50-year frequency and a value of 77 miles per hour for the 100-year frequency. The increase in wind forces for a wind of 100-year frequency over the forces used in our analysis based on 75 miles per hour velocity is not significant. Our consideration is, therefore, reasonably based on 75 miles per hour basic wind. When such a wind is combined with deadload and the temperatures, as discussed in Chapters IV and V, it represents an extreme design condition. To arrive at winds and temperatures that represent normal conditions is more difficult; but after review of recent probability studies and climatological data 6/, 7/, 16/, it is our opinion that a basic wind velocity of 50 mph, a maximum temperature of 90° F, and a minimum temperature of 20° F, represent normal conditions.

On this basis, the safety factors for normal conditions do not govern for any stresses considered so far in our studies of modified designs or, in other words, are better than minimums recommended by the A.I.S.C.

or other recognized specifications. The minimum safety factors for the extreme condition are the ones that will require our greatest attention.

In the analysis we have carried out, the results of which are included in this report, secondary as well as primary stresses have been included where it is feasible to compute them. Furthermore, all reasonably possible loads have been considered. Therefore, we can say that many uncertainties inherent in ordinary design will not be present, which should influence our considerations to allow some reductions in safety factors.

In the following table, minimum safety factors are shown for various loading conditions at Stations 15 and 44 on the arch. Two alternative designs are shown at Station 15 based on the type of joint used at the corners of the transverse triangular frame, hinged or rigid. Only the hinged joint is considered at Station 44. Hinged joints are assumed similar to those shown on Drawings No. XOA-D-903 and 905, with no supporting strut for the extrados leg; rather, the distance between skin plates is to be increased as recommended in Chapter V. The rigid corner design at Station 15 is assumed to be similar to that detailed on Drawing No. XOA-D-908 with additional transverse stiffeners to be added to the outside skin plates as discussed on Sheets Vf 24 and 25. Furthermore, it is assumed the diaphragms and stiffeners between skin plates are made continuous as recommended on page 6, Chapter V; also that the 1/4-inch outside skin plate in the vicinity of Station 44 will be changed from stainless to stainless-clad steel.

The safety factors in the table are about as large as can be obtained with the present cellular type of construction. As can be seen, they are small, particularly at Station 44, and extensive changes are required to obtain even these factors. The hinged corner design is preferred to the rigid corner because the assumptions relative to this design are better. We will point out just two factors:

First, reliance is placed on the use of a strut to maintain stresses less than yield point for the rigid corner design, but the strut is not to be used in the hinged corner design. In Chapter V it is shown how difficult it is to predict the effect the newly added strut has on the extrados side of the transverse triangular frames. A computation at Station 15 shows that if the strut is installed with a discrepancy of 1/8 inch from theoretical value, it affects some bending moments on the frame as much as 21 percent. It would be impossible to predict how much the strut may vary from theoretical, but it is quite possible that the discrepancy may be greater than the 1/8 inch assumed.

Second, it is also pointed out in Chapter V that the stresses in rigid-type corners are very complex. There are secondary effects of bending and shear and locked-in stresses due to welding which are superimposed on what are considered normal stresses. These secondary effects will vary with each individual joint depending on the way a joint is welded up. The joints are complicated weldments, difficult to make, and must be done by hand welding so that each joint depends on the individual making it, which in turn affects the secondary stresses mentioned above. Furthermore, some of the main stress-carrying welds are not only difficult to make but difficult to examine when made and impossible to examine in the future after the job is completed.

We mentioned that extensive changes are necessary to achieve even these small safety factors, which in our opinion are too small for a permanent monumental structure of this nature. We arrive at this conclusion after carefully weighing the preceding factors plus the fact that danger from loss of life is small. There still remain many uncertainties, particularly the use of an untried spot weld upon which the integrated action of inner and outer skin plates depends.

At the present time the sections of the arch above Station 45 that are already fabricated must be changed to replace the 1/4-inch stainless steel skin plate by a stainless-clad plate. This will necessitate a complete refabrication of these sections. Since this is necessary, in our opinion, it will be highly desirable to substitute another type of design as shown on Drawing No. XOA-D-913, and discussed in Chapter V.

The above safety factors are based on assumed static wind forces. Wind tunnel tests on the aerodynamic stability of the arch are presently being prepared. These will probably be far enough advanced in June 1965 to give specific answers to this problem. Therefore, all the above factors cannot be considered final, but await the outcome of the wind tunnel tests.

SAFETY FACTORS

Minimum safety factors for modification No. 2			
Case	Station 15		
	Rigid joint	Principal stresses	Buckling
A1	2.38	1.95	1.63
A2	2.92	1.68	1.40
B1	2.91	1.77	2.02
B2	2.91	1.76	1.45
C1	3.31	2.31	2.26
C2	4.15	1.95	1.64

Minimum safety factors for hinged frames						
Case	Station 15			Station 44		
	Hinged joint	Principal stresses	Buckling	Hinged joint	Principal stresses	Buckling
A1	1.85	1.89	1.97	2.10	2.25	2.33
A2	1.46	1.42	1.35	2.40	1.60	1.38
B1	2.21	1.79	1.49	2.84	2.09	1.96
B2	1.47	1.37	1.42	2.48	1.38	1.24
C1	2.55	2.21	1.93	3.05	2.49	2.43
C2	1.80	1.66	1.58	2.92	1.89	1.64

Chapter IV

ARCH--GENERAL ANALYSIS

1. General

The arch was analyzed as a statically indeterminate structure with forces, moments, and stresses evaluated in accordance with definitions and notations and at locations shown on Drawing No. X-OA-D-899.

The basic arch indeterminate analysis was solved by the stiffness method of analysis using matrix algebra techniques. A general purpose IBM-7090 computer program was developed for statically indeterminate structural analysis. Computer programs were also developed and used for computing section properties of the arch; magnitude of wind loads; and calculation of stresses, including creep, shrinkage, and post-tensioning effects.

Results of all computer programs, including the basic arch indeterminate analysis calculations, were spot checked by desk calculator. Derivation of formulas and development of computer programs are discussed in Appendix F.

2. Basic Data and Assumptions

The arch geometry, structural dimensions, and other basic data are as shown on the latest drawings and data made available to us by the National Park Service and their designer-consultants 1/, 2/, 3/.

The following assumptions are made for the analysis:

The arch in its completed form was analyzed in accordance with definitions and notations and at locations shown on Drawing No. X-OA-D-899.

For symmetrical load cases, 52 arch segments assumed.

For nonsymmetrical load cases, 26 arch segments assumed.

Arch stresses act in a longitudinal direction normal to typical section. See Drawing No. X-OA-D-899.

Normal stresses vary linearly across the section; a linear variation may be assumed with good accuracy since the ratio of arch radius to depth across arch section is large.

Arch sectional properties for deflection and normal stress computations include the effect of the longitudinal diaphragms and stiffeners.

Foundation yielding effects are not considered; that is, the arch is assumed fixed at the base.

Arch geometry is based on reference 2/.

Arch structural dimensions are based on reference 3/.

Modulus of elasticity of A-7 carbon and Type 304 stainless steel = 29×10^6 psi.

Modulus of elasticity of post-tensioning steel = 30×10^6 psi.

Modulus of elasticity of concrete = 5×10^6 psi.

Poisson's ratio = 0.25 (average value--entire arch).

Creep of concrete is based on the effective modulus method of analysis.

A creep factor of 1.5 is used for creep due to dead load of cantilever and post-tensioning forces.

A creep factor of 0.5 is used for creep due to closure forces.

Shrinkage strain of concrete is assumed as 0.000,170. This is estimated to be the sum of mass concrete shrinkage equal to 0.000,100 plus autogeneous shrinkage of 0.000,070.

Coefficient of thermal expansion per degree Fahrenheit:

Weighted value--arch above Station 45 = 0.000,007,1

Weighted value--arch below Station 45 = 0.000,005,5

Concrete = 0.000,005,5

Carbon steel = 0.000,006,5

Type 304 stainless steel = 0.000,009,6

3. Loading Assumptions

(a) Dead Load

In addition to direct weights of structure, this heading includes effect of post-tensioning, creep, shrinkage and closure force.

(a-1) Direct dead load

Steel skin and stiffeners at 490 lb/cu ft 3/.

Transverse triangular frames at 490 lb/cu ft 3/.

Interior framing and stairs at 490 lb/cu ft 3/.

Concrete (including post-tension and reinforcement steel)
at 150 lb/cu ft 3/.

(a-2) Closure force at crown of arch = 642.5 kips 3/

Reduction in closure force due to creep strain of the concrete is considered only where it increases maximum stresses.

(a-3) The post-tensioning bars are assumed initially stressed to 142 kips each. The number of bars are those shown on the latest drawings 3/.

(a-4) Creep

The creep of concrete is based on the effective modulus method of analysis. A creep factor dependent on strength and modulus of elasticity of concrete and the age of loading is used. An average value of 1.5 was used for dead load and 0.5 for the closure force.

(a-5) Shrinkage strain of concrete assumed as 0.000,170

(a-6) Discussion of creep and shrinkage effects

The two accepted methods of creep analysis are, first, the effective modulus method and, second, the superposition method. The superposition method provides a more rational step-by-step procedure but requires accurate information on the physical and creep properties of the actual concrete used as well as the construction program giving detailed placement dates and post-tensioning schedules. Since this information was not available, we used the effective modulus method, using an average creep factor of 1.5 for dead load creep. Considerable variation in the assumed creep factor showed little variation in creep stresses. A creep factor value of 0.5 was used for creep effect due to crown closure force.

(b) Wind Loads

(b-1) Basic wind load assumptions used in the analyses are:

South wind** (wind in plane of arch) as shown on Drawing No. X-OA-D-900; west wind** (wind normal to plane of arch) as shown on Drawing No. X-OA-D-901; and southwest wind** (wind vectors point 30° north of east) as shown on Drawing No. X-OA-D-902.

(b-2) Discussion of wind load assumptions

Loads shown on the drawings are a product of the dynamic pressure due to the wind, a shape factor, an inclination factor, and the width of the arch.

Dynamic pressure is determined from a basic wind velocity of 75 miles per hour, 30 feet above ground. This corresponds to the value at St. Louis on the 50-year frequency map 5/, 18/. For heights above 30 feet, the wind velocity is varied from that at a 30-foot height by the 1/7-power formula

$V_z = 75 \left(\frac{z}{30} \right)^{1/7}$. This wind velocity is then increased by a gust factor of 1.1.

The shape factors, a drag coefficient for forces parallel to the wind and a lift coefficient for forces normal to the wind, varied depending on the direction of the wind relative to a section through the arch. These were $C_D = 1.8$ and $C_L = 0$ for wind normal to a side of a triangular section; $C_D = 1.1$ and $C_L = 0$ for wind directed toward the apex of a triangular section; $C_D = 1.8$ and $C_L = 1.0$ for wind parallel to a side of a triangular section. See page 12, Appendix A.

The inclination factor used was $\frac{2 \sin A}{1 + \sin^2 A}$, where A is the angle between the wind direction and the centroidal axis of the arch.

For detailed discussion of wind forces, see Appendix A.

(b-3) Recorded wind velocities

The maximum recorded wind velocity for 45 years of record at St. Louis is given as 82 mph occurring in March. See reference 7/. A value of 73 mph is given for September, and from March through November every month shows a value of 60 mph or greater, except July which reveals 56 mph. The wind direction of these maximum values was from the southwest in most

**West wind indicates wind from the west; south wind indicates wind from the south; southwest wind indicates wind from the southwest.

cases. The prevailing wind direction is shown as south on an annual basis; on a monthly basis it is from the south May through November, west February through April, and northwest in December and January.

Prior to the 45-year period discussed above, a maximum velocity of 91 mph was recorded May 27, 1896, the same day as one of the most disastrous tornadoes in history. Weather Bureau records 17/ show that the frequency of tornado occurrences in the St. Louis area is relatively high. It is recognized that there is a possibility of tornado occurrence and that peripheral wind velocities of tornadoes are frequently estimated in excess of 300 mph. It is not realistic to design the structure to resist the forces due to such very extreme wind velocities, but rather utilize the tornado forecasting facilities of the Weather Bureau to give advance warning to evacuate people from the arch or from any dangerous position relative to it.

(c) Temperature

For the portion of the arch above Station 45, the following assumptions are made: a shop fabrication temperature of 70° F is assumed for all conditions of temperature drop; a shop fabrication temperature of 40° F is assumed for all conditions of temperature rise.

For the composite steel and concrete portion of the arch below Station 45, the important temperature is that existing at the time the concrete sets sufficiently to develop adequate bond strength so that the concrete and steel function as a unit.

For the case of temperature drop, a placement temperature of 80° F is assumed plus a 20° F rise due to heat of hydration resulting in a maximum initial temperature of 100° F.

For the case of temperature rise, a placement temperature of 40° F is assumed plus an estimated rise of 10° F due to heat of hydration resulting in a minimum initial temperature of 50° F.

A temperature of 60° F at arch closure is assumed for calculation of temperature stresses in the arch indeterminate structure analysis.

The interior temperature used in the thermal stress analyses is based on the assumptions that the heating design temperature is 65° F and the cooling design temperature is 75° F.

Metal temperatures in the exterior and interior skin plates for use in the design are based on climatological records 6/ for the

St. Louis area and on heat transfer studies discussed in Appendix B. An approximate check of calculated metal temperatures and the temperature gradient through the wall under solar exposure was obtained experimentally as described in Appendix D.

U.S. Weather Bureau reference 6/ gives the following recorded temperatures at St. Louis:

Maximum temperature = 115° F July 14, 1954

Temperature of 100° F or higher expected 5 days per year

Temperature of 90° F or higher expected 46 days per year

Minimum temperatures recorded by U.S. Weather Bureau =
 -14° F January 19, 1940

All-time low at St. Louis = -23° F January 1864

Temperature of zero or below expected 2 days per year

(d) Earthquake Loads

The St. Louis area is active seismically, and based on historical records spanning 150 years, it is considered proper to design the arch for an earthquake of intensity MM VI-VII (modified Mercalli intensity VI-VII) the maximum recorded.

The earthquake loads due to an earthquake of this magnitude have been determined by the spectral response concept using the fundamental modes of vibration of the arch in and normal to the plane of the arch and using acceleration spectra for three earthquakes in this intensity range.

The resulting earthquake loads are shown in Appendix C and are of a magnitude considerably less than those for wind loads. A detailed analysis of arch stresses due to earthquake loading was therefore not made.

Refer to Appendix C for detailed discussion of earthquake loads.

(e) Live Loads

The observation platform load and equipment loads are symmetrical live loads 3/, 20/.

(e-1) Observation platform load

Load intensity on observation platform is 100 lb/ft^2 .

Observation platform load = 0.743 k/ft (from crown to Station 5).

(e-2) Train in lower portion of leg

Concentrated force of 40k parallel to the centroidal axis at Station 6 halfway between Station 5 and Station 6.

(e-3) Train in upper portion of leg

Uniformly distributed load of 0.3 k/ft from Station 3 to Station 13.

(e-4) Equipment loads

The following two loads are fixed loads once the arch is completed. Since they do not occur until near or at completion, they were considered as live loads.

1. Train hoist weight

Concentrated load of 12k acting vertically downward halfway between Station 5 and Station 6.

2. Motor generator weight

Concentrated load of 10k acting vertically downward halfway between Station 2 and Station 3.

4. Assumed Loading Combinations

The effect of dead load, including post-tensioning, creep and shrinkage of concrete, and a closure force of 642.5 kips , is considered in all loading cases. The magnitude of wind velocities and exterior air temperatures assumed in the following loading combinations were selected after consideration of all available data and review of actual recorded values and study of climatological records and probability data 5/, 6/, 7/, 16/, 18/. Metal temperatures in the exterior and interior steel plates used in the design were established by heat transfer studies presented in Appendix B. In the indeterminate analysis of the arch, an average uniform temperature variation only is considered. In addition to this, it is necessary to consider the important effect of the temperature variation between

the inside and outside steel skin plates in evaluating thermal stresses both in longitudinal and transverse directions. The effect of solar energy is included on the extrados side of the south leg.

The following loading conditions were assumed for the analyses. All temperatures are in Fahrenheit degrees.

Loading Conditions A--Extreme Temperature

Case A1 Extreme Temperature Drop

Wind velocity at 30 ft 25 mph

Outside air temperature -10

Inside air temperature 60

	Above Station 45	Station 46	Station 71
Temperature--exterior steel plate. .	-5	-5	-6
Temperature--interior steel plate. .	55	35	47

Case A2 Extreme Temperature Rise

Wind velocity at 30 ft 60 mph

Outside air temperature 114

Inside air temperature 90

	Above Station 45	Station 46	Station 71
Temperature--exterior steel plate			
Extrados side (south leg)	145	135	135
Other sides	110	112	113
Temperature--interior steel plate			
Extrados side (south leg)	95	99	97
Other sides	90	98	94

Loading Conditions B--Extreme Wind

Case B1 Extreme Wind with Temperature Drop

Wind velocity at 30 ft 75 mph

Outside air temperature. 20

Inside air temperature 65

Above
Station 45 Station 46

Temperature--exterior steel plate. . 20 25

Temperature--interior steel plate. . 60 50

Case B2 Extreme Wind with Temperature Rise

Wind velocity at 30 ft 75 mph

Outside air temperature. 95

Inside air temperature 75

Above
Station 45 Station 46

Steel temperatures--exterior plate

Extrados side (south leg) 125 115

Other sides 94 93

Steel temperatures--interior plate

Extrados side (south leg) 80 90

Other sides 77 81

Loading Conditions C--Normal Design Load

Case C1 Normal Wind with Temperature Drop

Wind velocity at 30 ft 50 mph
 Outside air temperature. 20
 Inside air temperature 65

Above
Station 45 Station 46

Temperature--exterior steel plate. .	20	25
Temperature--interior steel plate. .	60	50

Case C2 Normal Wind with Temperature Rise

Wind velocity at 30 ft 50 mph
 Outside air temperature. 90
 Inside air temperature 75

Above
Station 45 Station 46

Temperature--exterior steel plate

Extrados side (south leg)	125	115
Other sides	90	89

Temperature--interior steel plate

Extrados side (south leg)	80	90
Other sides	77	80

5. Results

(a) General

The results of the indeterminate structural analysis of the arch as presently designed include values of displacements, forces,

moments and normal stresses in accordance with definitions and at locations noted on Drawing No. X-OA-D-899.

Forces, moments, and stresses were computed for the following individual loads:

Dead load

Live load

Temperature (+50° F)

West wind load (75 mph basic wind load)

South wind load (75 mph basic wind load)

Southwest wind load (75 mph basic wind load)

Drawings No. X-OA-D-910 and X-OA-D-911 show the results for south wind load and west wind load, respectively. Dead and live loads stresses and the maximum longitudinal arch stresses above Station 45 are shown on Drawing No. X-OA-D-912. Inspection of the live load stresses indicates these stresses are not significant. Appendix F shows the computer analysis and results which give more complete information.

Torsional stresses are not large provided that the continuity of the triangular-shaped skin plate sections is maintained. These stresses reach a maximum for west or southwest wind load conditions but do not exceed 2,500 psi, based on the assumption of uniform distribution of shear stress along the sides of the arch and through both the 1/4-inch exterior plate and the 3/8-inch interior plate. In view of the relatively low shear stress intensity, we considered it unnecessary to make the more complex shear flow analysis which would be required for a more rigorous determination of stresses in the cellular-type structure of the arch wall.

Also, in view of their relatively small magnitude, these shear stresses have been neglected in the determination of the maximum longitudinal and transverse stresses in the steel plates. Consideration of these shearing stresses would result in principal stresses of slightly greater magnitude and slightly lower factors of safety.

(b) Displacements

The maximum displacement occurs at the crown of the arch and amounts to 26.7 inches for the west wind load. Arch displacements

due to south wind and west wind are plotted on Drawings No. X-OA-D-910 and X-OA-D-911.

A tabulation showing the magnitude of displacements at the crown of the arch due to the assumed basic wind loads follows:

Loading	Displacements in inches		
	X	Y	Z
West wind	0	-0.7	26.7
South wind	5.6	0.3	0
Southwest wind	2.7	-0.1	21.5

Positive displacements are referenced to coordinate axes shown on Drawing No. X-OA-D-899.

(c) Longitudinal Arch Stresses--above Station 45

The basic longitudinal arch stresses act normal to arch cross sections in accordance with notations shown on Drawing No. X-OA-D-899. Stresses in the exterior steel plate due to a south wind and due to a west wind are shown on Drawings No. X-OA-D-910 and X-OA-D-911, respectively.

The maximum longitudinal arch stresses in the exterior steel plate at selected arch stations with the corresponding loading combination are shown on Drawing No. X-OA-D-912. Longitudinal arch stresses at the interior plate are not shown since they are consistently lower. Values are given at four critical points of the triangular cross section shown. The indicated maximum longitudinal arch stress occurs at Station 44 and amounts to 29,100 psi compression for loading condition B2. The major contribution of thermal stresses to the maximum longitudinal arch stresses is shown in the tabulation on the drawing. For example, at location centerline EW at arch Station 15, the total longitudinal arch stress under loading conditions A2 is 17,800 psi compression of which the temperature stress amounts to 11,700 psi. It is important to recognize that these temperature stresses are not localized concentrations but extend at this intensity range across the side of the arch. Also, at Station 15 at the location designated EW1 where the total longitudinal stress under loading condition A2 is 22,600 psi compression, the temperature stress amounts to 16,400 psi.

The transverse stresses accompanying the maximum longitudinal arch stresses are also shown on Drawing No. X-OA-D-912. These transverse stresses are not the maximum but merely the transverse stress acting at the same location under the same loading condition. In fact, the maximum transverse stresses are higher than longitudinal stresses and exceed the yield point at several locations at the interior steel plate and are discussed subsequently in Chapter V. The thermal stresses also contribute a major part of the transverse stresses.

Combinations of longitudinal arch stresses and transverse stresses exist that are more severe than shown in the tables on Drawing No. X-OA-D-912, particularly for cases where one stress is compression and the other is tension. For further discussion of this, refer to Chapter V.

Thermal stresses listed on Drawing No. X-OA-D-912 were computed by digital computer program developed by the basic methods set forth in Chapter 14 of "Theory of Elasticity" by Timoshenko and Goodier 19/. An independent check by desk calculator methods was obtained as shown in Appendix B. The latter is given in some detail to permit the reader to make as detailed a check as he wishes.

(d) Longitudinal Arch Stresses--below Station 45

Stresses which develop in the lower portion of the arch are influenced to a considerable degree by post-tensioning loads and creep and shrinkage of the concrete. Accurate knowledge of the loads and scheduling of the post-tensioning operation and the creep and shrinkage properties of the concrete, particularly the creep as related to age of concrete at loading, are very important in evaluating stresses due to these effects. As no information was made available to us on a detailed post-tensioning schedule 22/ or creep properties of the concrete, we proceeded using such general information we had received plus our own research and information from our laboratories to reach a judgment on values to use in our creep analysis.

The analysis was first made assuming a fully effective composite section including the post-tensioning steel, the uncracked concrete section, the conventional reinforcement, as well as the structural steel skin plates.

The following tabulation shows the longitudinal arch stresses due to dead load which includes the effects of the crown closure force, post-tensioning forces, and creep and shrinkage of concrete. These values were obtained by use of the Effective

Modulus method with a creep factor of 1.5 for dead load and 0.5 for crown closure force and a shrinkage strain of 0.000,170.

Longitudinal Arch Stresses--Dead Load
(exterior plate effective)
all stresses in psi
+ = tension - = compression

Station	Exterior Steel Plate		Exterior Concrete Face	
	Intrados	Extrados	Intrados	Extrados
45	-4000	-14000	+520	-820
53	-5800	-13100	+350	-730
61	-7600	-12100	+190	-640
71	-9100	-12900	+40	-670
Station	Interior Steel Plate		Interior Concrete Face	
	Intrados	Extrados	Intrados	Extrados
45	-4700	-13700	+430	-770
53	-6400	-12800	+260	-680
61	-8100	-11900	+100	-600
71	-9600	-12600	-50	-620

Inspection of the above stresses discloses the maximum stresses occur at or just below Station 45, with a compression in the exterior stainless steel plate of 14,000 psi at the extrados and a tension of 520 psi in the concrete at the intrados.

For the extreme wind loading condition B2, the maximum arch stress including dead load occurs in the exterior steel plate just below Station 45 and amounts to 18,300 psi compression at the extrados. The maximum tensile stress in the concrete occurs at the intrados of the same station and for the same loading condition and amounts to 850 psi. Maximum compression in the concrete is 1,600 psi and occurs at Station 71.

Under normal loading conditions C2, the maximum compression in the steel plate is 16,000 psi at the extrados, and maximum tension in the concrete is 680 psi at the intrados. These stresses also occur just below Station 45.

Thermal gradient effects are excluded from the above.

Excessive stresses will develop at the welds at the exterior stainless steel plate due to temperature, creep, and shrinkage, see Appendix B. Repetition of stresses of such a magnitude may lead to failure of these welds and separation of the exterior plate from the concrete, hence resulting in its ineffectiveness as a part of the composite section.

Therefore, longitudinal arch stresses were analyzed assuming the outside skin plate ineffective. The results of this analysis show the following stresses.

Longitudinal Arch Stresses--Dead Load
(exterior plate not effective)
all stresses in psi
+ = tension - = compression

Station	Exterior Steel Plate		Exterior Concrete Face	
	Intrados	Extrados	Intrados	Extrados
45	-----	-----	+510	-960
53	-----	-----	+320	-840
61	-----	-----	+150	-740
71	-----	-----	-10	-740
Station	Interior Steel Plate		Interior Concrete Face	
	Intrados	Extrados	Intrados	Extrados
45	-5700	-15500	+410	-920
53	-7500	-14200	+220	-790
61	-9100	-13000	+60	-680
71	-10600	-13600	-100	-790

Comparison of the values in the above tables shows that when the exterior plate is not effective the maximum tension in the concrete is reduced from 520 to 510 psi at the intrados but that interior plate compressive stresses are increased about 10 percent. The maximum interior steel plate stress is 15,500 psi at Station 45 at the extrados.

Results of tests of cylinders of arch concrete given in Appendix D indicate the 28-day strengths in excess of the design strength of 5,000 psi. Based on the formula that ultimate tensile strength equals $5 \sqrt{f'_c}$, a value of 350 psi is indicated. In view of the uncertainty of the behavior of concrete in tension and the deep and unusual type section of the arch, we believe this value should not be exceeded.

Since values exceeding this are indicated in the region just below Station 45 at the intrados, cracking of the concrete is indicated in this area. With the exterior plate not effective and the concrete cracked, the maximum interior steel plate stress for dead load conditions is approximately 16,000 psi.

Assuming the exterior plate effective but the concrete cracked at the intrados just below Station 45, the exterior steel plate longitudinal stress at the extrados is about 17,000 psi for the dead load condition.

6. Stresses Using Wind Loads Assumed in Original Design

The following tabulation shows the stresses determined by computer analysis for Stations 15 and 45 using the wind load assumption of Severud et al. 1/, 8/. Also shown for comparison are the corresponding stresses determined using Bureau of Reclamation wind load assumptions. The lower stresses indicated by the Severud wind load assumption are due to the fact that the Severud loads are minimum values based on a general shape factor of 1.3, which represents "the effect of the combined inward and outward pressures on exterior surfaces of ordinary rectangular buildings"; see page 11, Reference 8.

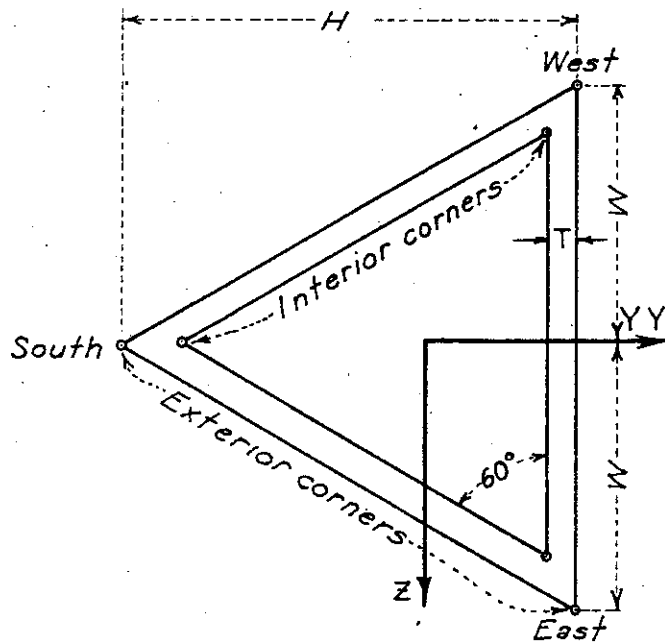
Longitudinal Arch Stresses--Wind Load Only
all stresses in psi
+ = tension - = compression

Station	Wind load direction	Point of stress	Wind Load Assumption	
			Severud	USBR*
15	South	Intrados	-4400	-7900
		Extrados	+1900	+3600
	North	Intrados	+4700	+8200
		Extrados	-2200	-4000
	West	South	-----	-300
		East	-100	+300
		West	+100	+700
45	South	Intrados	+2800	+5100
		Extrados	-1600	-2700
	North	Intrados	-2700	-5200
		Extrados	+1600	+2600
	West	South	-----	+100
		East	-8900	-12100
		West	+8900	+12400

*Values are shown on Drawings No. X-OA-D-910 and X-OA-D-911.

7. Effect of Number of Segments Assumed in Arch Analysis

The effect of the number of arch segments assumed in the analysis on the accuracy of the results was evaluated. During the preliminary studies and development of the computer program for the indeterminate structural analysis, an 18-segment arch was assumed, that is, 9 segments for each half of the arch. Comparison of the results of this 18-segment arch with those using the 52-segment arch disclosed differences of less than 2 percent in the significant values.



TYPICAL SECTION A-A

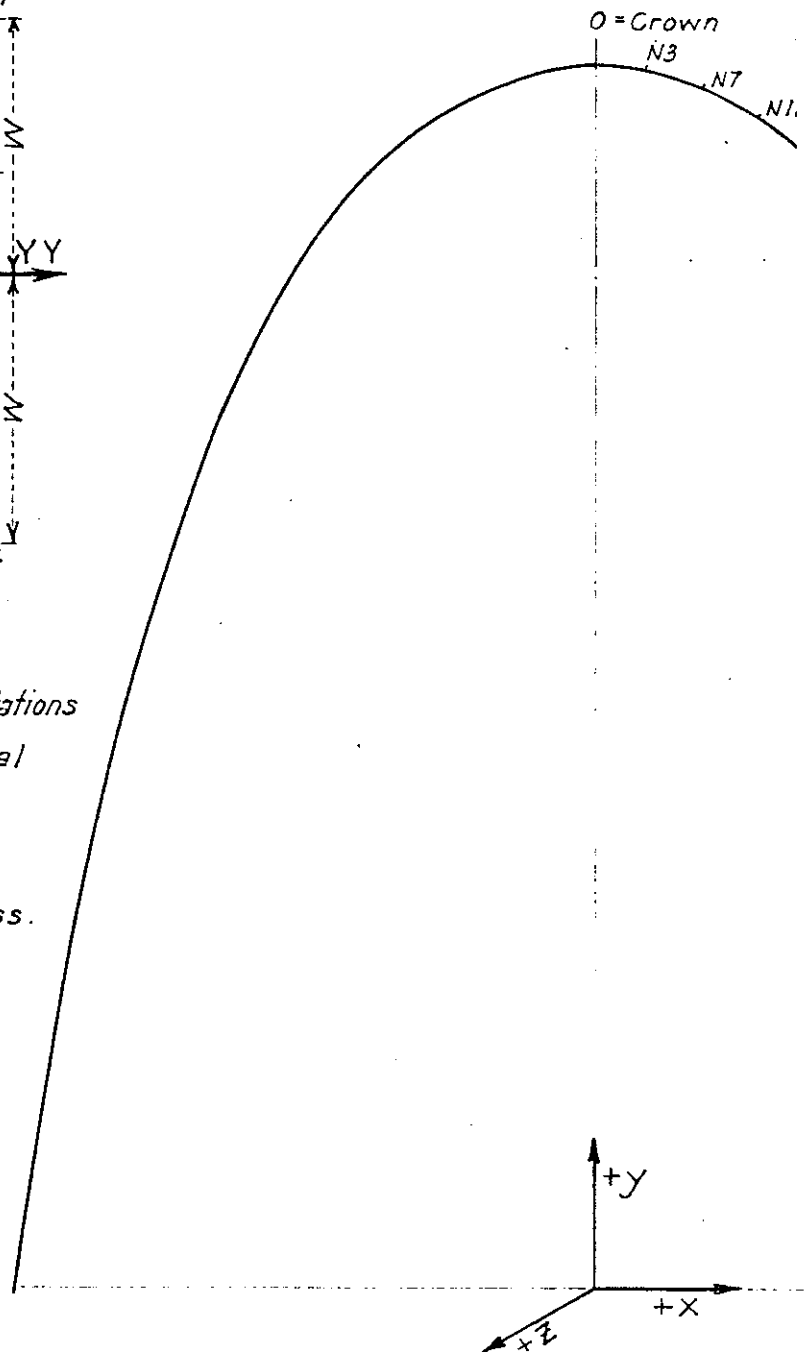
Normal to centroidal axis

Locations for Normal Stress Computations

- Indicates location where normal stress is computed.

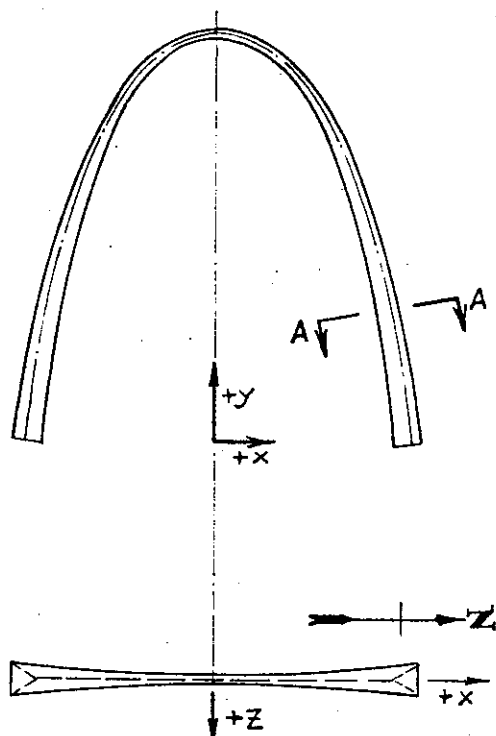
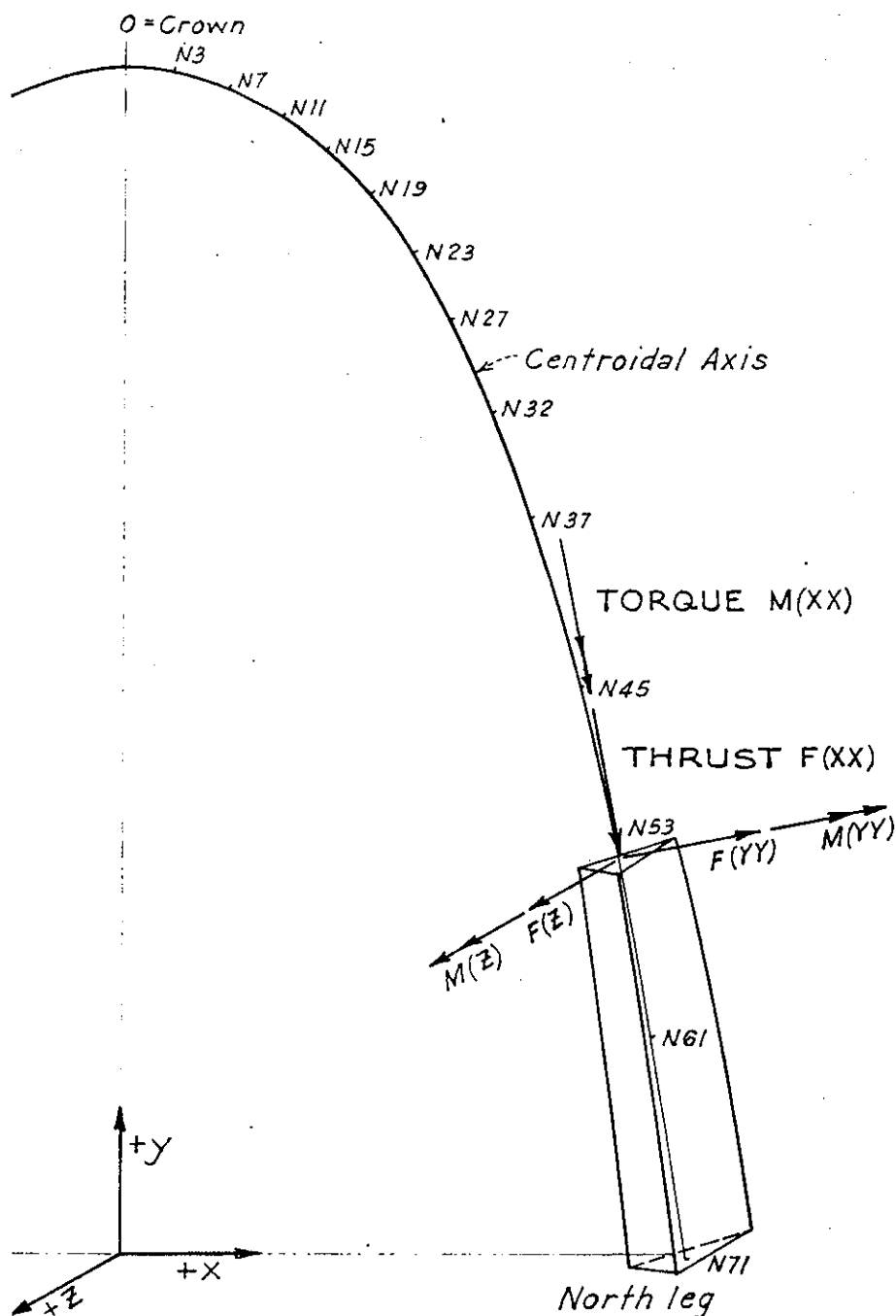
Sign convention:

- + Indicates tensile stress.
- Indicates compressive stress.



Moments and Forces refer to principal Axes of typical section

Moment and Force Vector shown in their positive direction
 → indicates —



NOTES

N3, N7, N11 etc. indicates stations used in analysis. eg N15 indicates station 15 north leg.

Analyses for all loading conditions are computed for north leg only. For non-symmetrical loading conditions stresses in south leg can be obtained as follows:

For stresses and forces in south leg due to south wind, see results of analysis of north leg due to north wind. For stresses and forces in south leg due to southwest wind see results of analysis of north leg due to northwest wind.

Stresses and Forces referenced to Centroidal Axis of typical section.

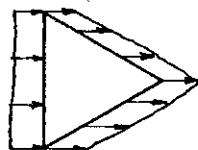
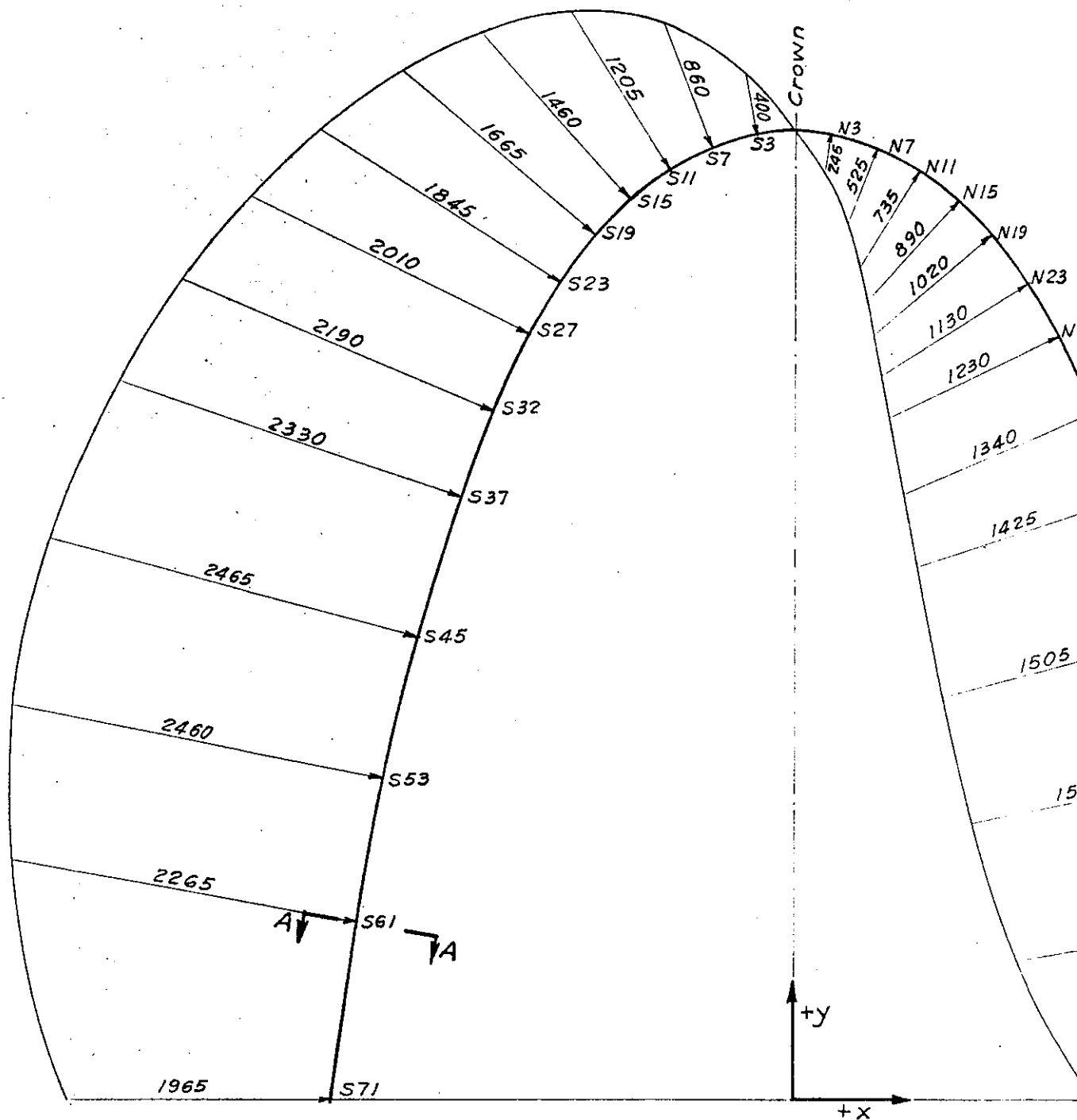
Force Vectors are in their positive directions.

→ indicates \ominus

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

JNEM GATEWAY ARCH MOMENTS, FORCES AND STRESSES SIGN CONVENTIONS AND DEFINITIONS

DRAWN *MSF* SUBMITTED
TRACED RECOMMENDED
CHECKED *PK* APPROVED
DENVER, COLO., JUNE 4, 1964 X-0A-D-899



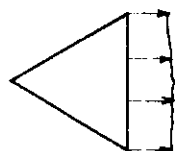
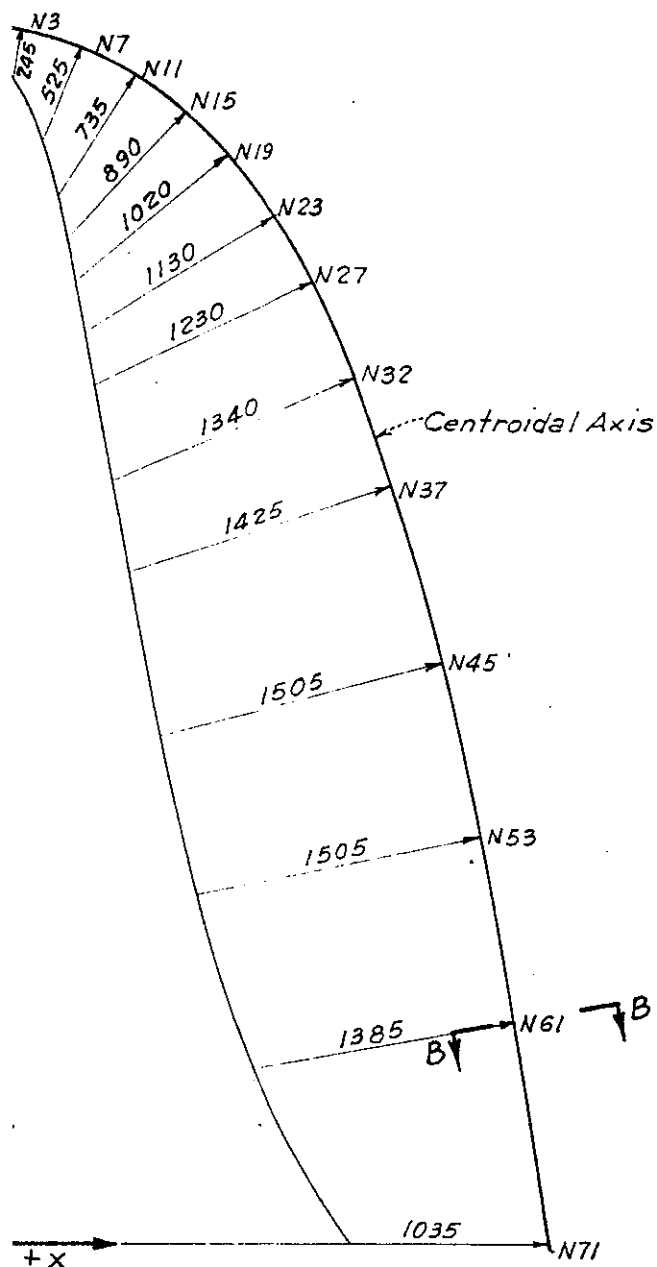
Total force component
= 2265*/ft. →

TYPICAL SECTION A-A

FORCE COMPONENTS IN PLANE OF ARCH

Tot

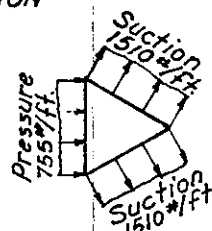
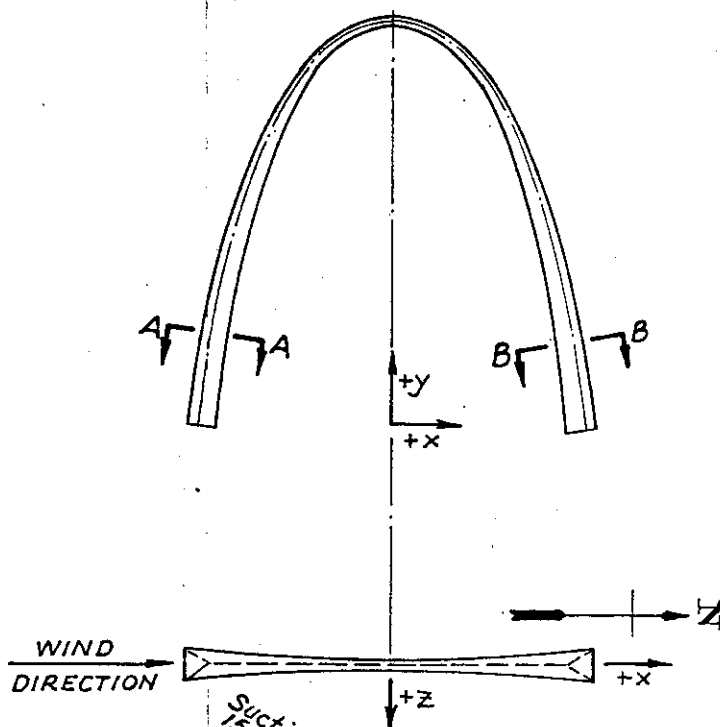
TY



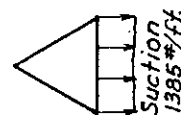
Total force component
= 1385#/ft. →

TYPICAL SECTION B-B

PLANE OF ARCH



SECTION A-A



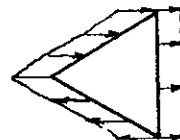
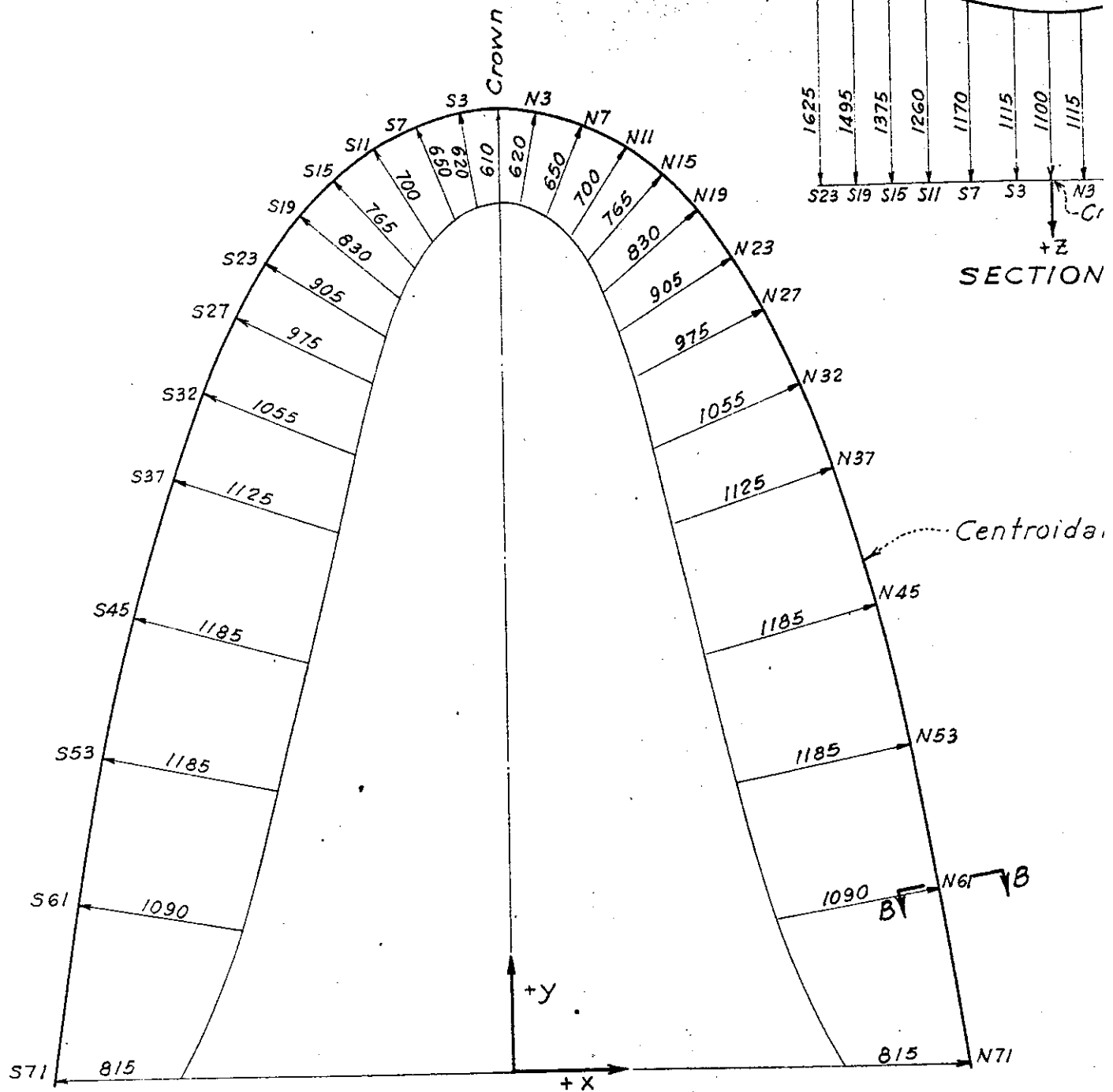
SECTION B-B

TOTAL FORCES ON TYPICAL SECTIONS

NOTES

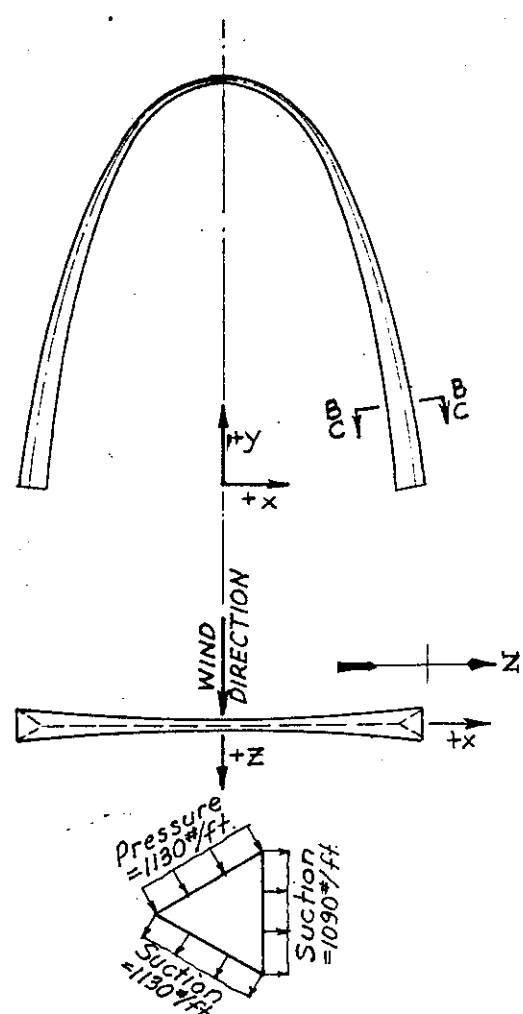
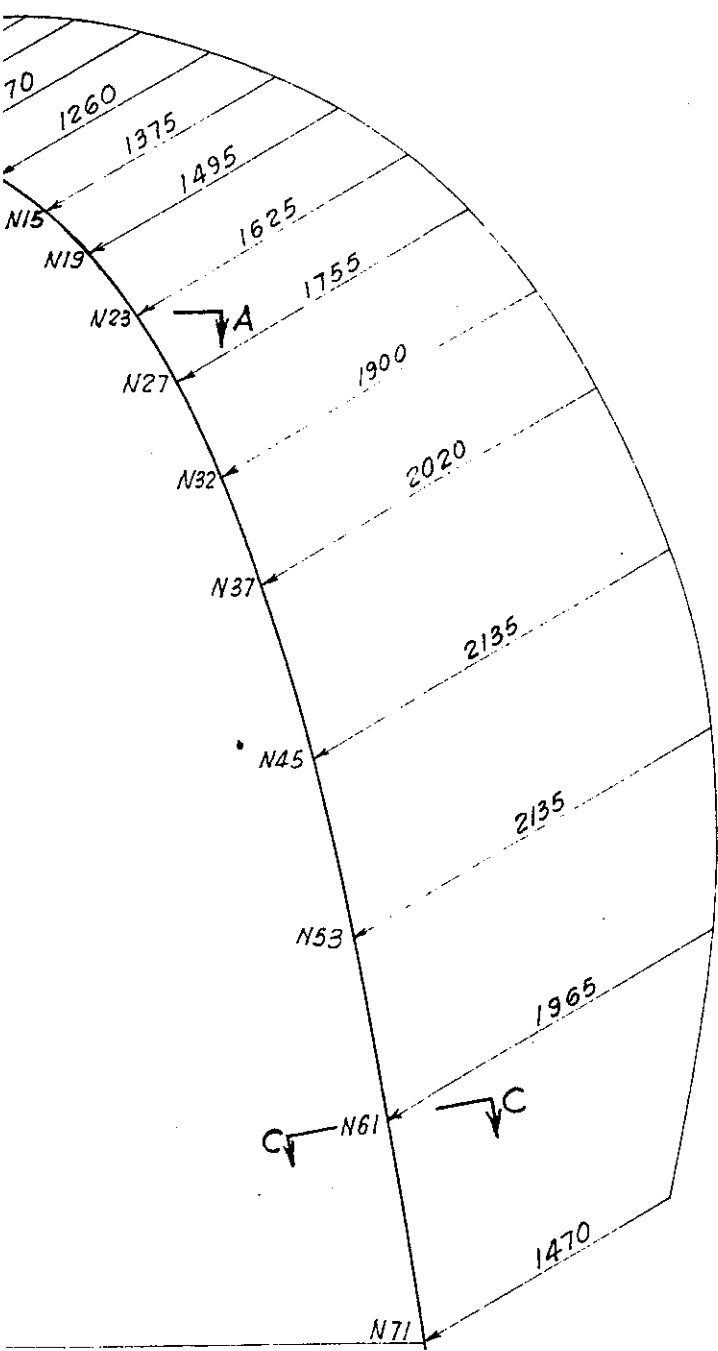
Forces are in lbs. per lineal ft. along the centroidal axis.
A total force component is equal to the algebraic sum of the pressure and suction components on all faces.

UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	
JNEM GATEWAY ARCH	
EXTERNAL FORCE COMPONENTS DUE TO SOUTH WIND	
DRAWN <i>MCF</i>	SUBMITTED <i>Edwin Rose</i>
TRACED	RECOMMENDED
CHECKED <i>2/3</i>	APPROVED
DENVER, COLO., JUNE 4, 1964 X-0A-D-900	



Total force component
= 1090 $\frac{\text{lb}}{\text{ft}}$ \rightarrow
TYPICAL SECTION B-B

FORCE COMPONENTS IN PLANE OF ARCH

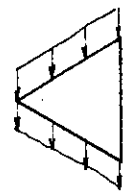


SECTION B-B
SECTION C-C

TOTAL FORCES ON TYPICAL SECTIONS

NOTES

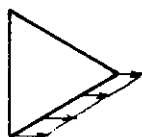
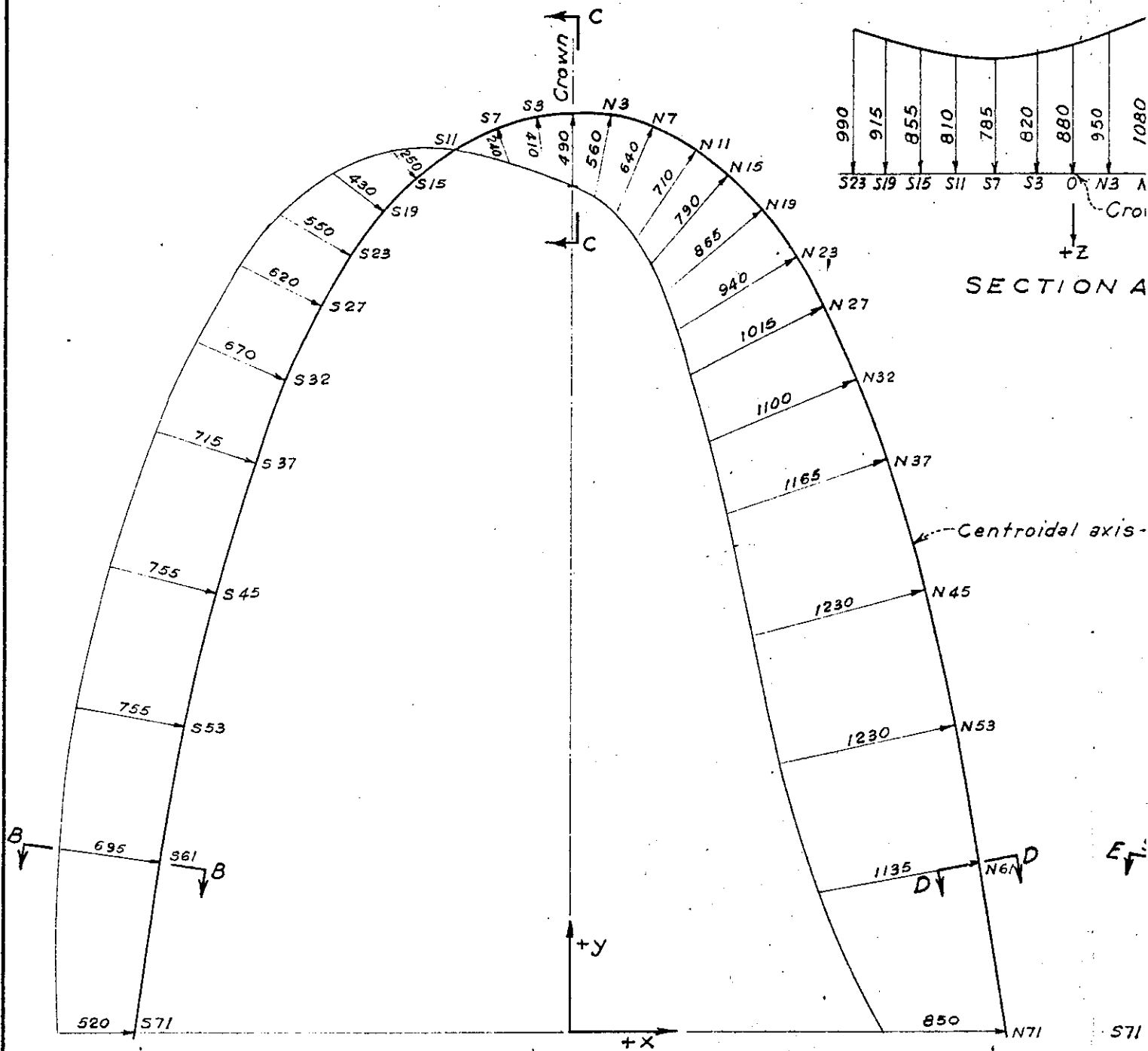
Forces are in lbs. per lineal ft. along the arch centroidal axis.
A total force component is equal to the algebraic sum of the pressure and suction components on all faces.



Total force component
= 1965 #/ft
TYPICAL SECTION C-C

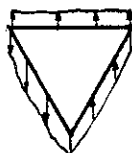
TO PLANE OF ARCH

UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	
JNEM GATEWAY ARCH	
EXTERNAL FORCE COMPONENTS DUE TO WEST WIND	
DRAWN MCF	SUBMITTED <i>Arthur Rose</i>
TRACED	RECOMMENDED
CHECKED <i>PK</i>	APPROVED
DENVER, COLO.,	JUNE 4, 1964 X-0A-D-901



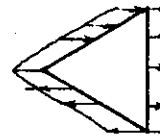
Total force component
= 695*/ft. →

TYPICAL SECTION B-B



Total force component
= 490*/ft. ↑

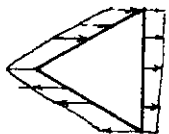
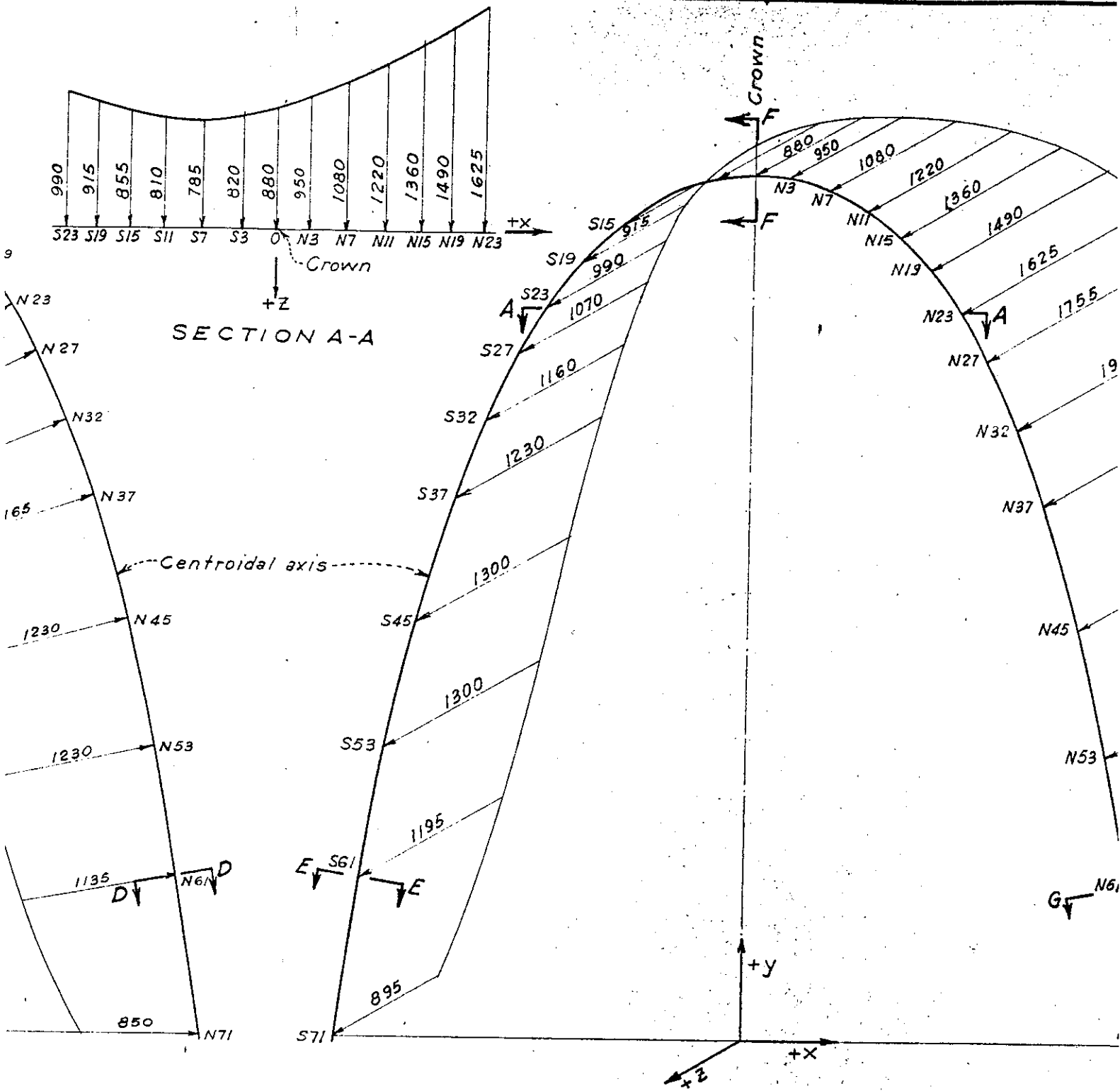
TYPICAL SECTION C-C



Total force component
= 1135*/ft. →

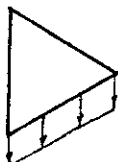
TYPICAL SECTION D-D

FORCE COMPONENTS IN PLANE OF ARCH



Total force component
= 1135#/ft. →

TYPICAL SECTION D-D



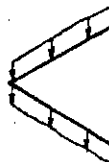
Total force component
= 1195#/ft. ↓

TYPICAL SECTION E-E



Total force component
= 880#/ft. ←

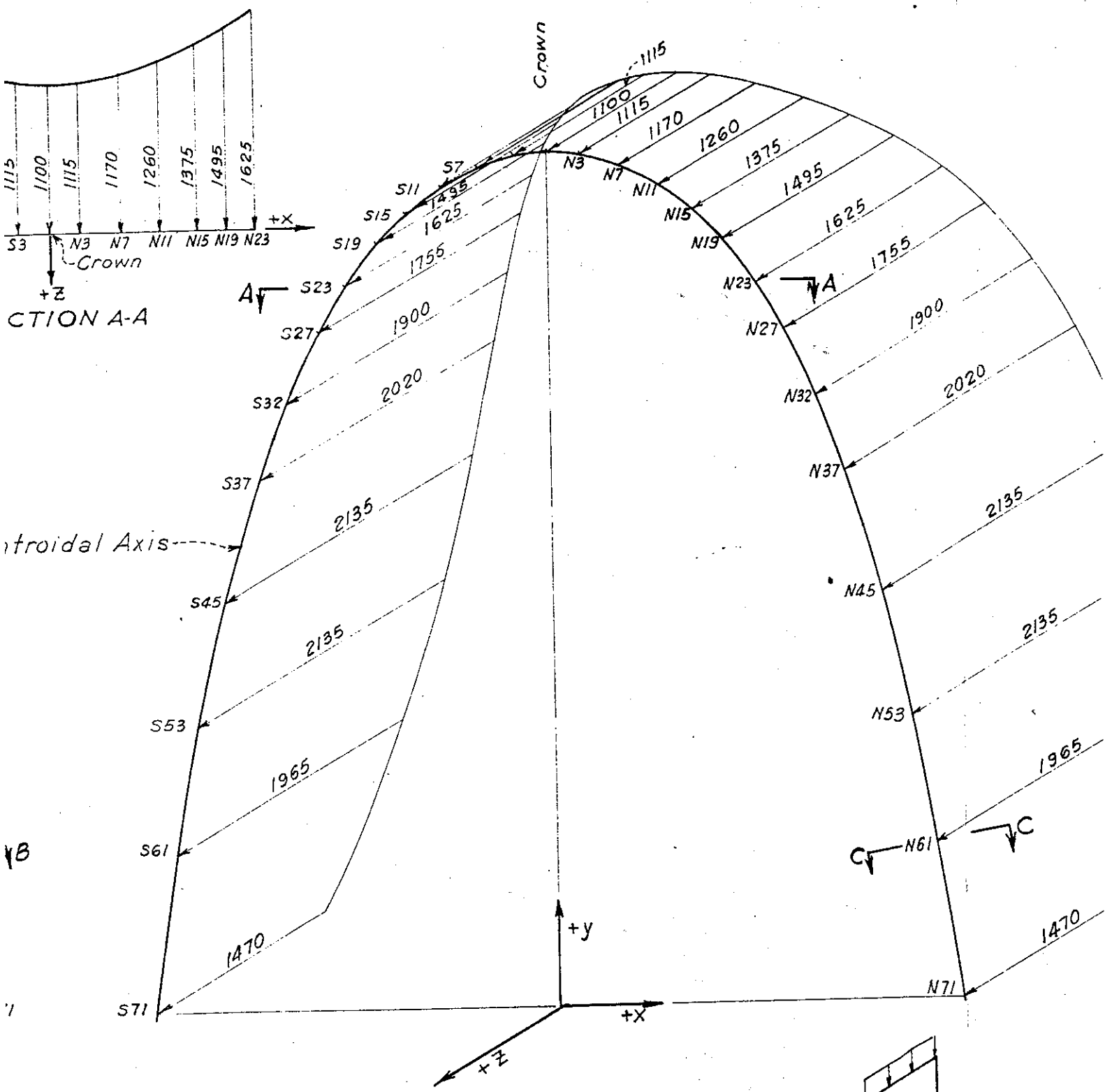
TYPICAL SECTION F-F



Total force component
= 1965#/ft. →

TYPICAL SECTION G-G

FORCE COMPONENTS NORMAL TO PLANE OF ARCH



SECTION A-A

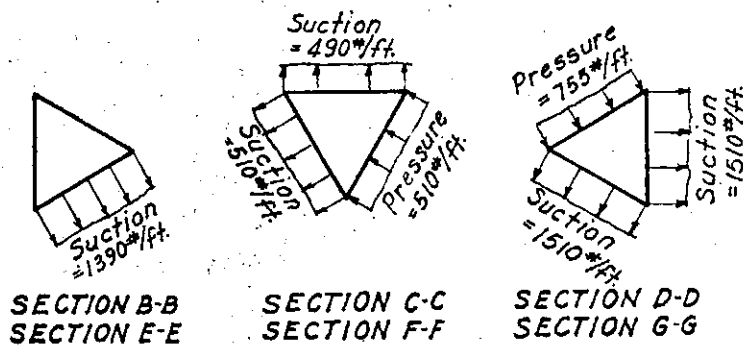
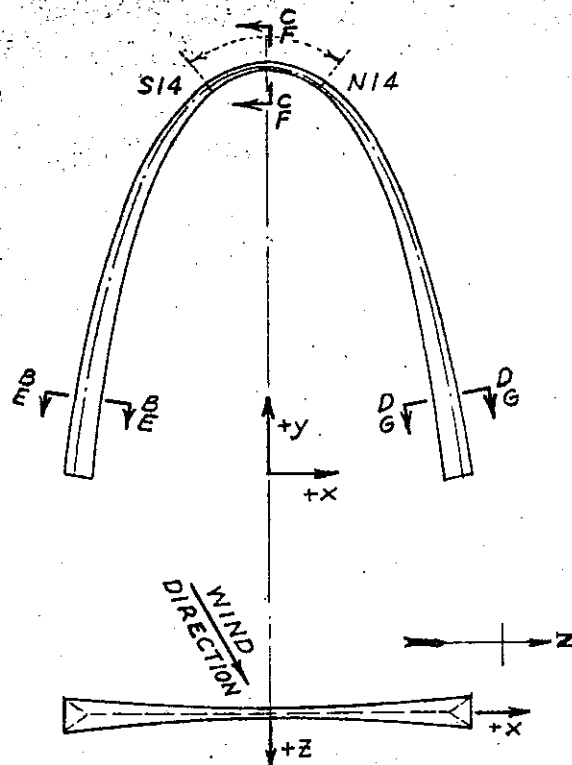
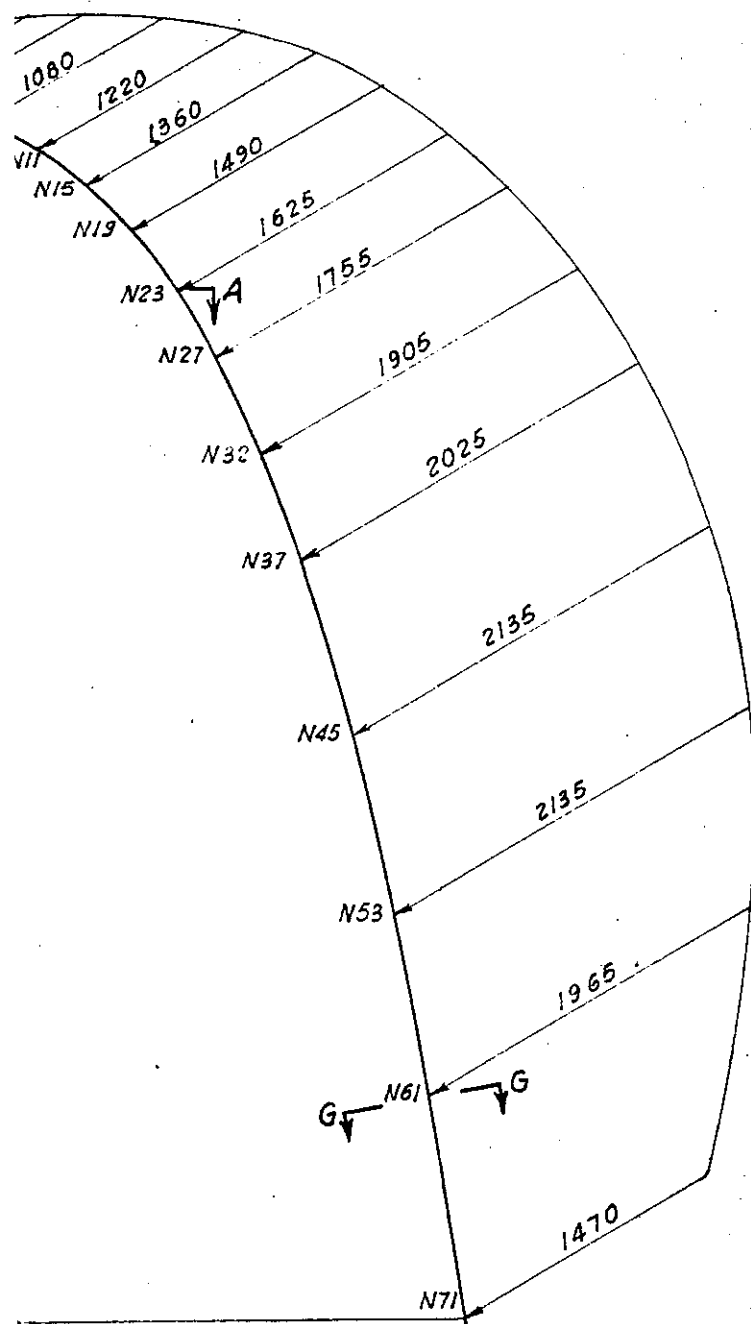
Centroidal Axis

YB

N B-B

Total force component
= 1965*/ft
TYPICAL SECTION C-C

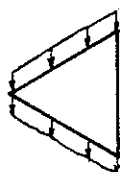
FORCE COMPONENTS NORMAL TO PLANE OF ARCH



TOTAL FORCES ON TYPICAL SECTIONS

NOTES

Forces are in lbs. per lineal ft. along the arch centroidal axis.
A total force component is equal to the algebraic sum of the pressure and suction components on all faces.

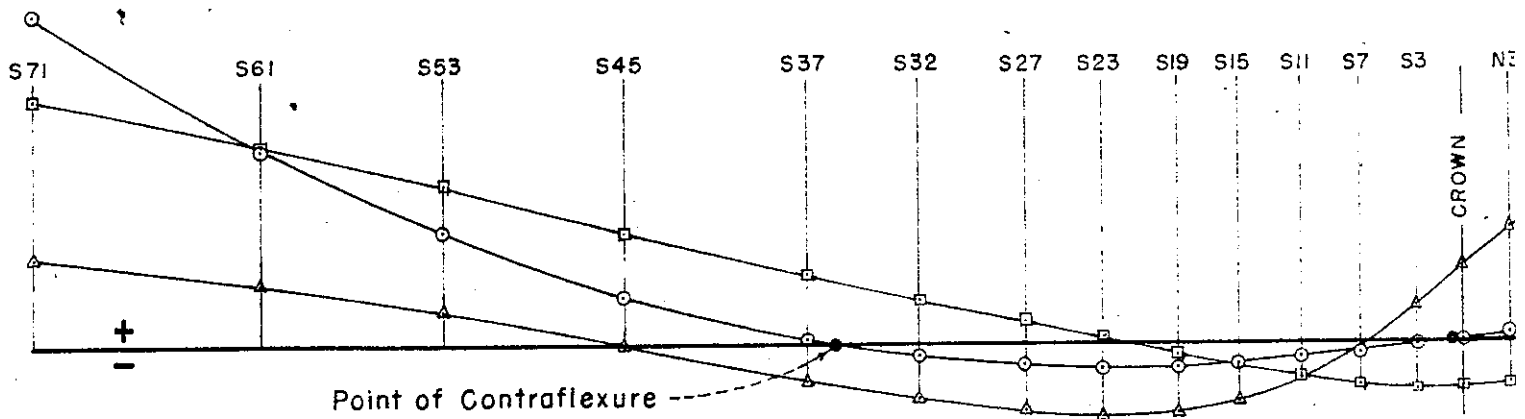


Total force component
= 1965#/ft.

TYPICAL SECTION G-G

PLANE OF ARCH

UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	
JNEM GATEWAY ARCH	
EXTERNAL FORCE COMPONENTS DUE TO SOUTHWEST WIND	
DRAWN MCF	SUBMITTED <i>Edwin Rose</i>
TRACED	RECOMMENDED
CHECKED <i>019</i>	APPROVED
DENVER, COLO., JUNE 4, 1964 X-0A-D-902	

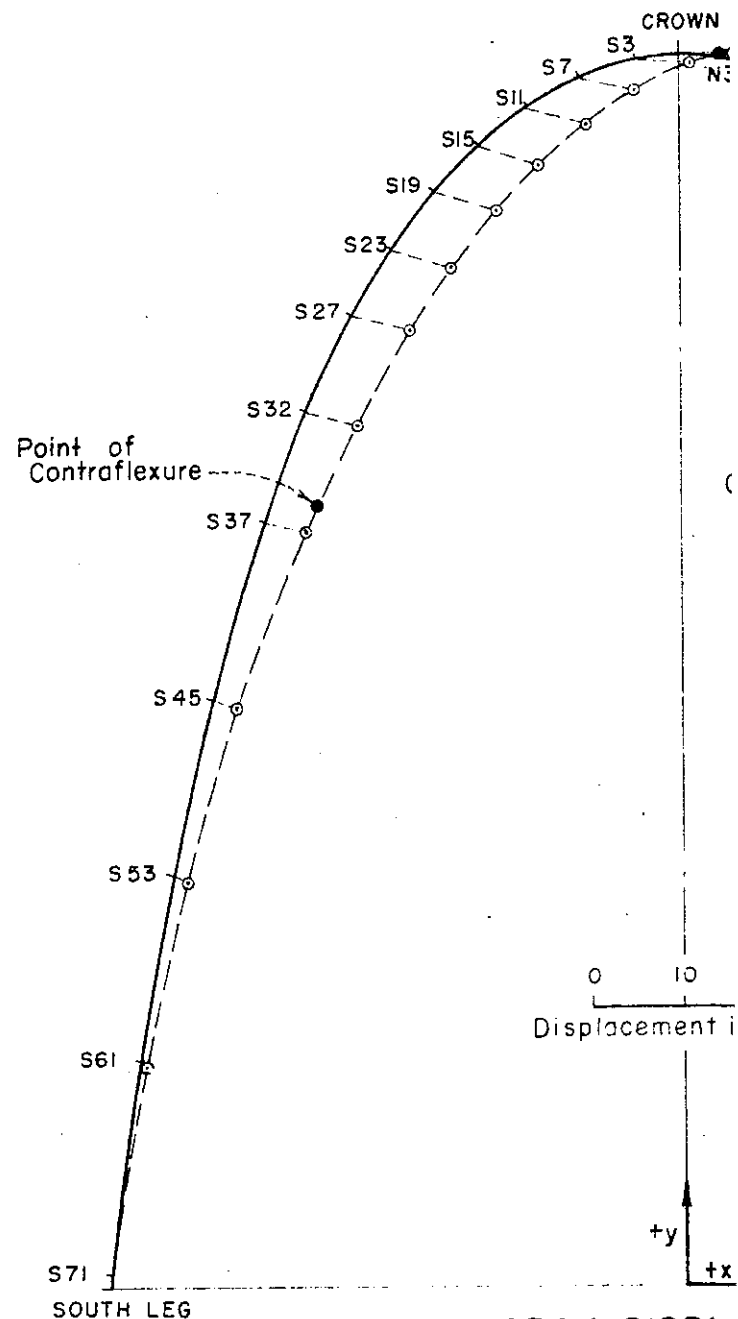


ARCH MOMENTS AND FORCES-PLOTTED

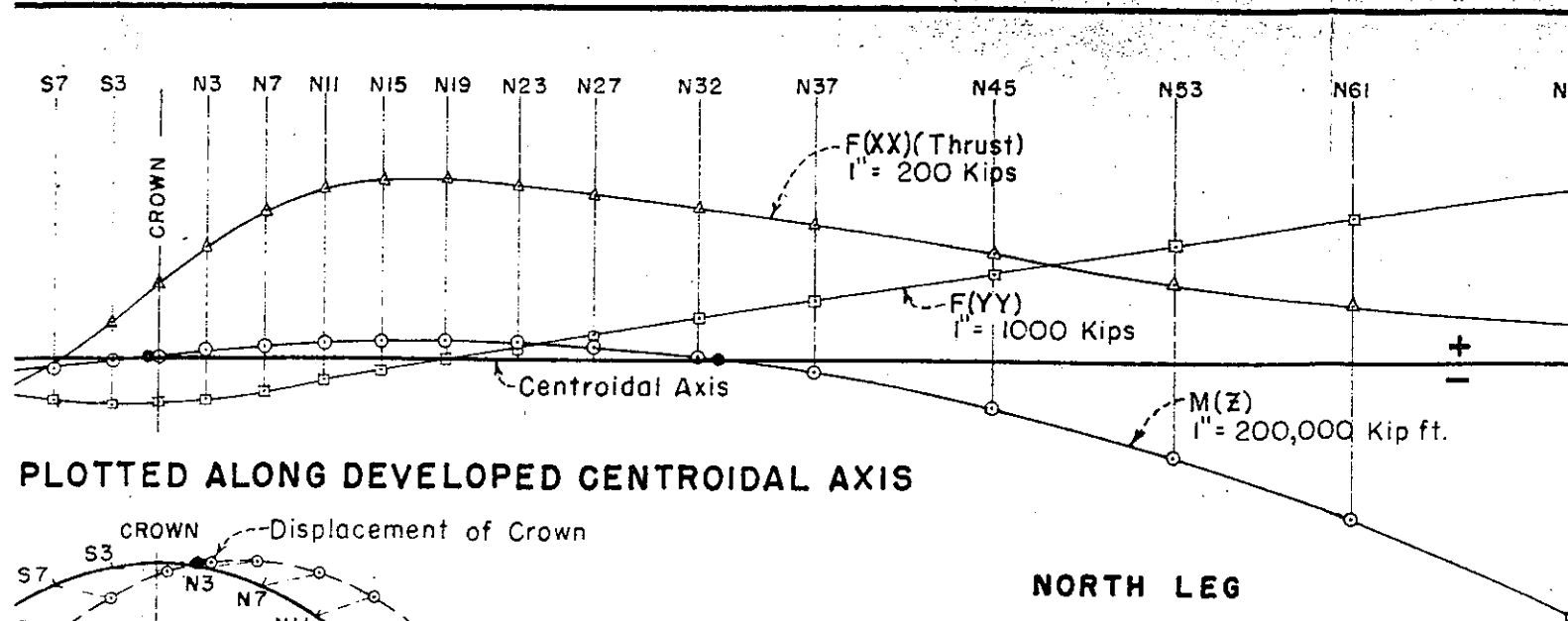
SOUTH LEG

Arch Sta.	Extrados		Intrados
	E	W	N
CROWN	581	581	-1585
S3	-1074	-1074	1939
S7	-3225	-3225	6414
S11	-4011	-4011	8189
S15	-3995	-3995	8222
S19	-3816	-3816	7884
S23	-3075	-3075	6383
S27	-2197	-2197	4582
S32	-987	-987	2111
S37	308	308	-539
S45	2596	2596	-5189
S45	909	909	-1820
* S45 C	156	156	-313
S53	1404	1404	-2823
* S53 C	242	242	-487
S61	1748	1748	-3516
* S61 C	301	301	-606
S71	2032	2032	-4085
* S71 C	350	350	-704

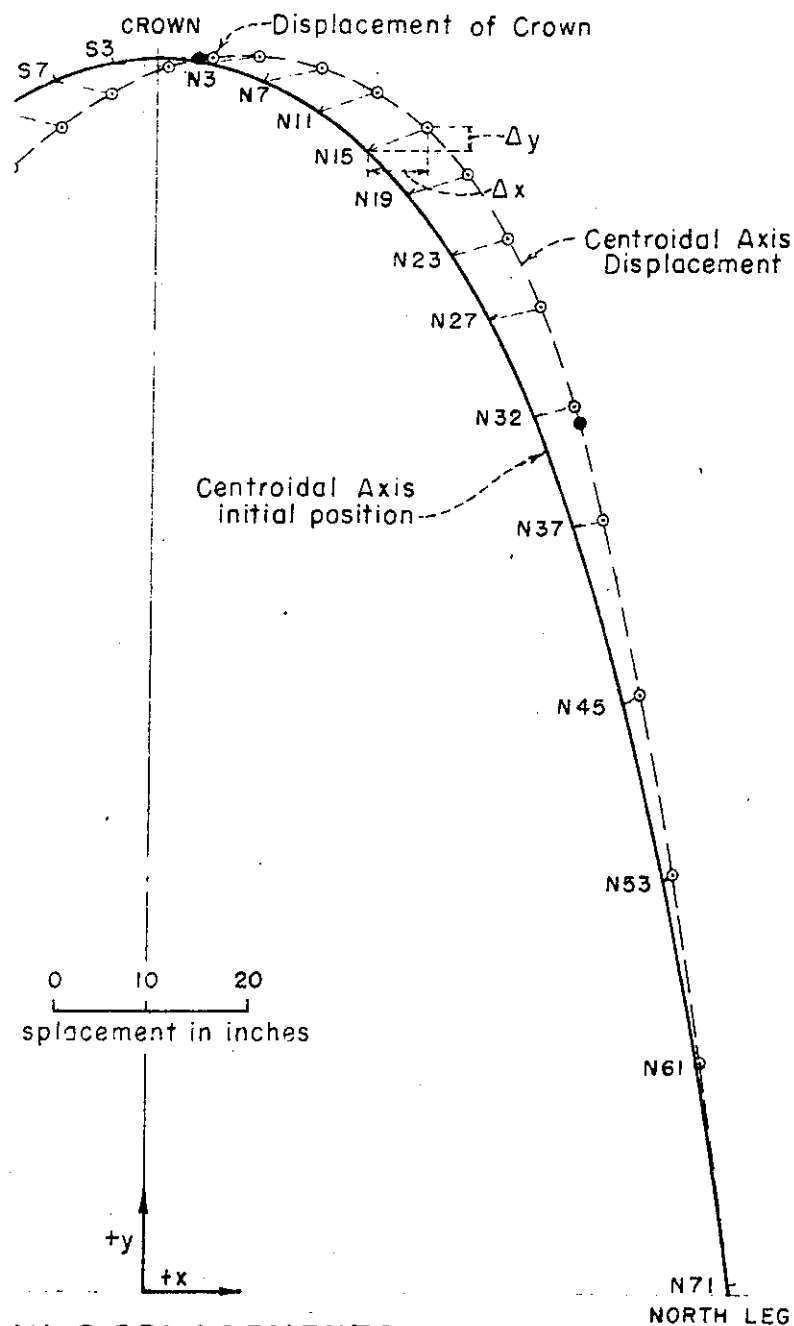
* Indicates stress in concrete.



ARCH DISPL



PLOTTED ALONG DEVELOPED CENTROIDAL AXIS

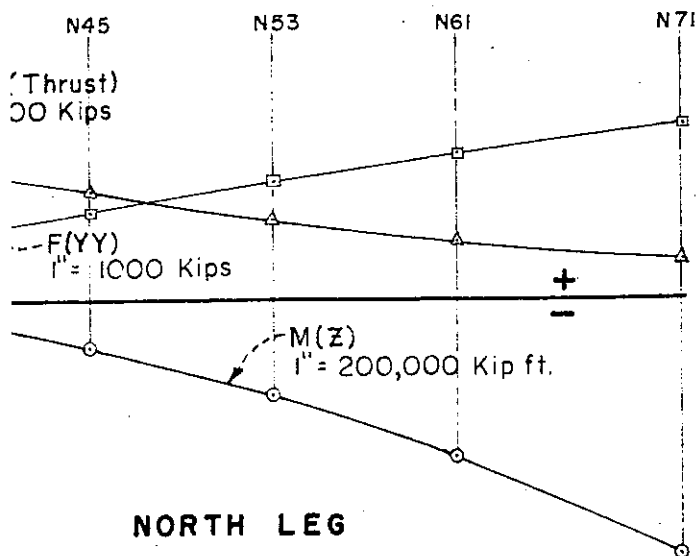


NORTH LEG

NORMAL STRESSES			
Arch Sta.	Intrados	Extrados	
	S	E	W
CROWN	- 1585	581	581
N3	- 4698	2042	2042
N7	- 8407	3826	3826
N11	- 8829	4017	4017
N15	- 7854	3562	3562
N19	- 6791	3051	3051
N23	- 4922	2164	2164
N27	- 3025	1268	1268
N32	- 722	160	160
N37	1492	- 905	- 905
N45	5135	- 2678	- 2678
N45	1803	- 935	- 935
* N45 C	311	- 161	- 161
N53	2429	- 1233	- 1233
* N53 C	419	- 213	- 213
N61	2831	- 1426	- 1426
* N61 C	488	- 246	- 246
N71	3133	- 1572	- 1572
* N71 C	540	- 271	- 271

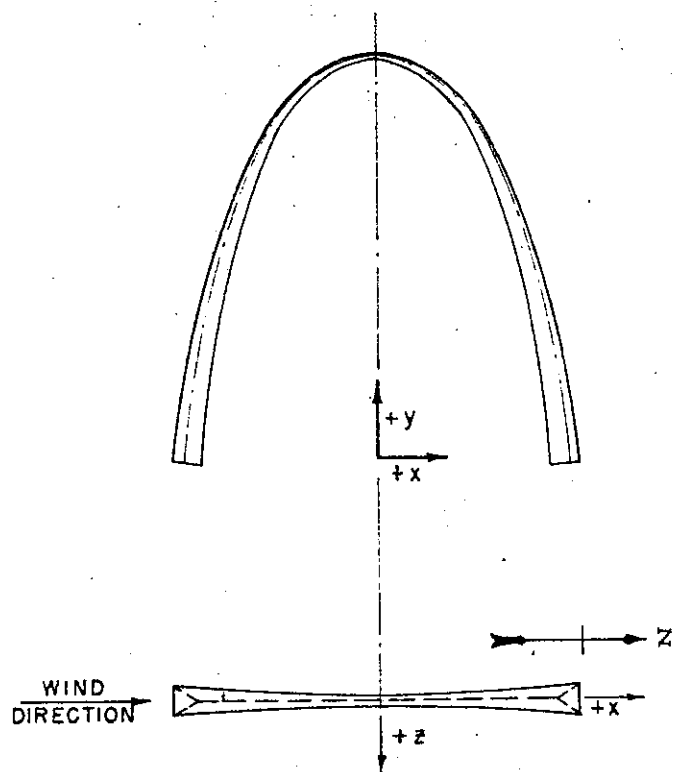
* Indicates stress in concrete.

CH DISPLACEMENTS



NORMAL STRESSES			
Arch Sta.	Intrados	Extrados	
	S	E	W
CROWN	- 1585	581	581
N3	- 4698	2042	2042
N7	- 8407	3826	3826
N11	- 8829	4017	4017
N15	- 7854	3562	3562
N19	- 6791	3051	3051
N23	- 4922	2164	2164
N27	- 3025	1268	1268
N32	- 722	160	160
N37	1492	- 905	- 905
N45	5135	- 2678	- 2678
N45	1803	- 935	- 935
* N45 C	311	- 161	- 161
N53	2429	- 1233	- 1233
* N53 C	419	- 213	- 213
N61	2831	- 1426	- 1426
* N61 C	488	- 246	- 246
N71	3133	- 1572	- 1572
* N71 C	540	- 271	- 271

* Indicates stress in concrete.



NOTES

For sign conventions and notations, see drawing No. X-OA-D-899.

Basic wind velocity - 75 mph at 30 feet above ground.

For external force components due to south wind, see drawing No. X-OA-D-900.

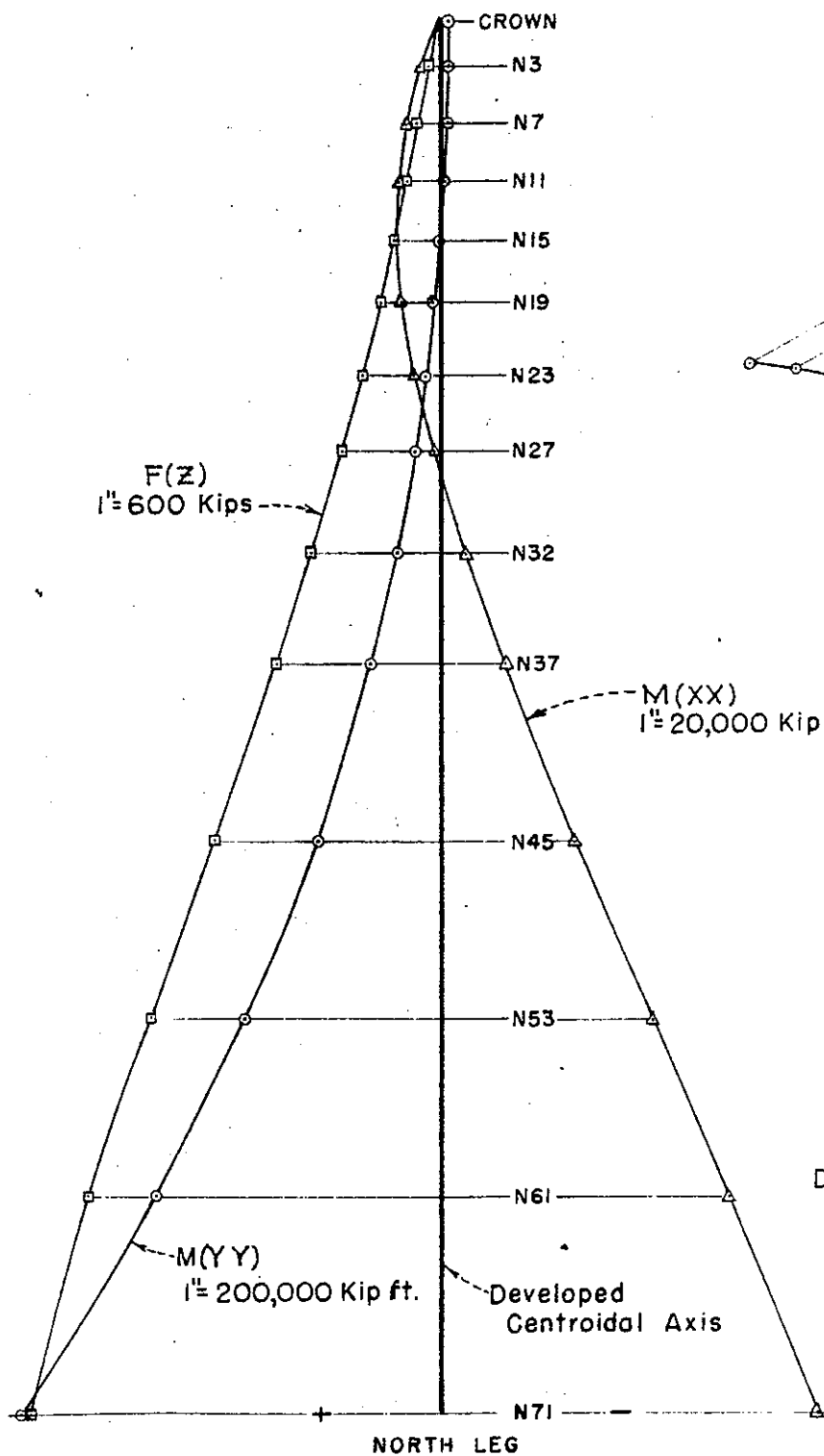
Stresses shown are those in exterior steel plate except locations marked with an asterisk (*) which indicate concrete stresses.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

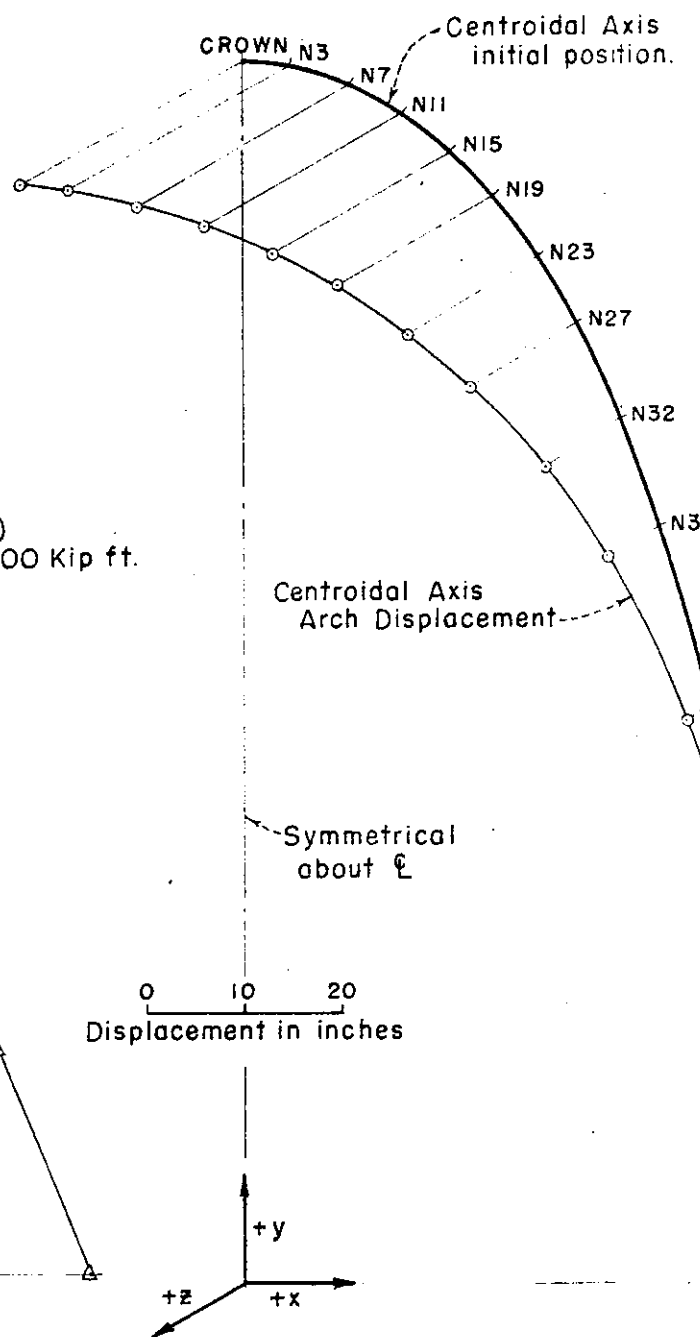
**JNEM - GATEWAY ARCH
MOMENTS, FORCES, STRESSES AND
DISPLACEMENTS - SOUTH WIND**

DRAWN G.H.H. SUBMITTED *C. Dunn Rose*
TRACED RECOMMENDED
CHECKED WNA APPROVED

DENVER, COLO., NOV. 20, 1964 X-OA-D-910



ARCH MOMENTS AND FORCES
M(XX) M(YY) F(Z)

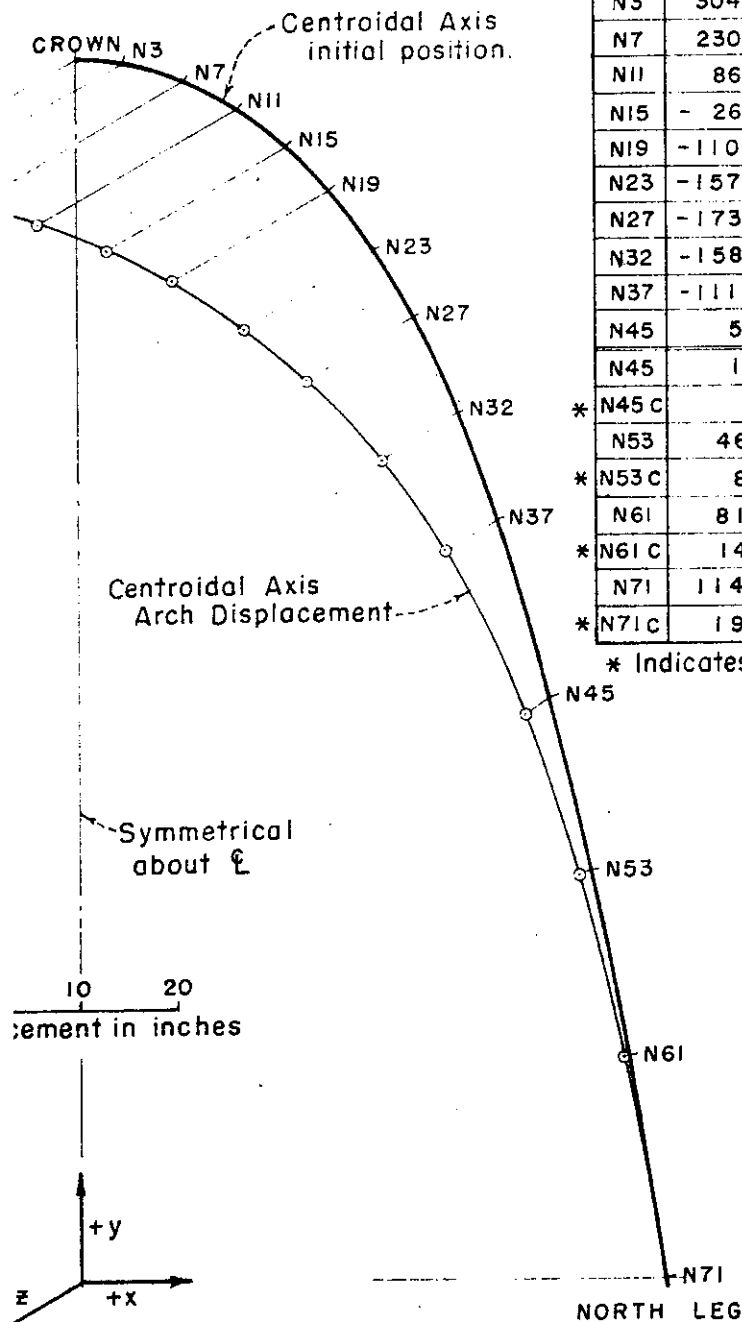


ARCH DISPLACEMENTS

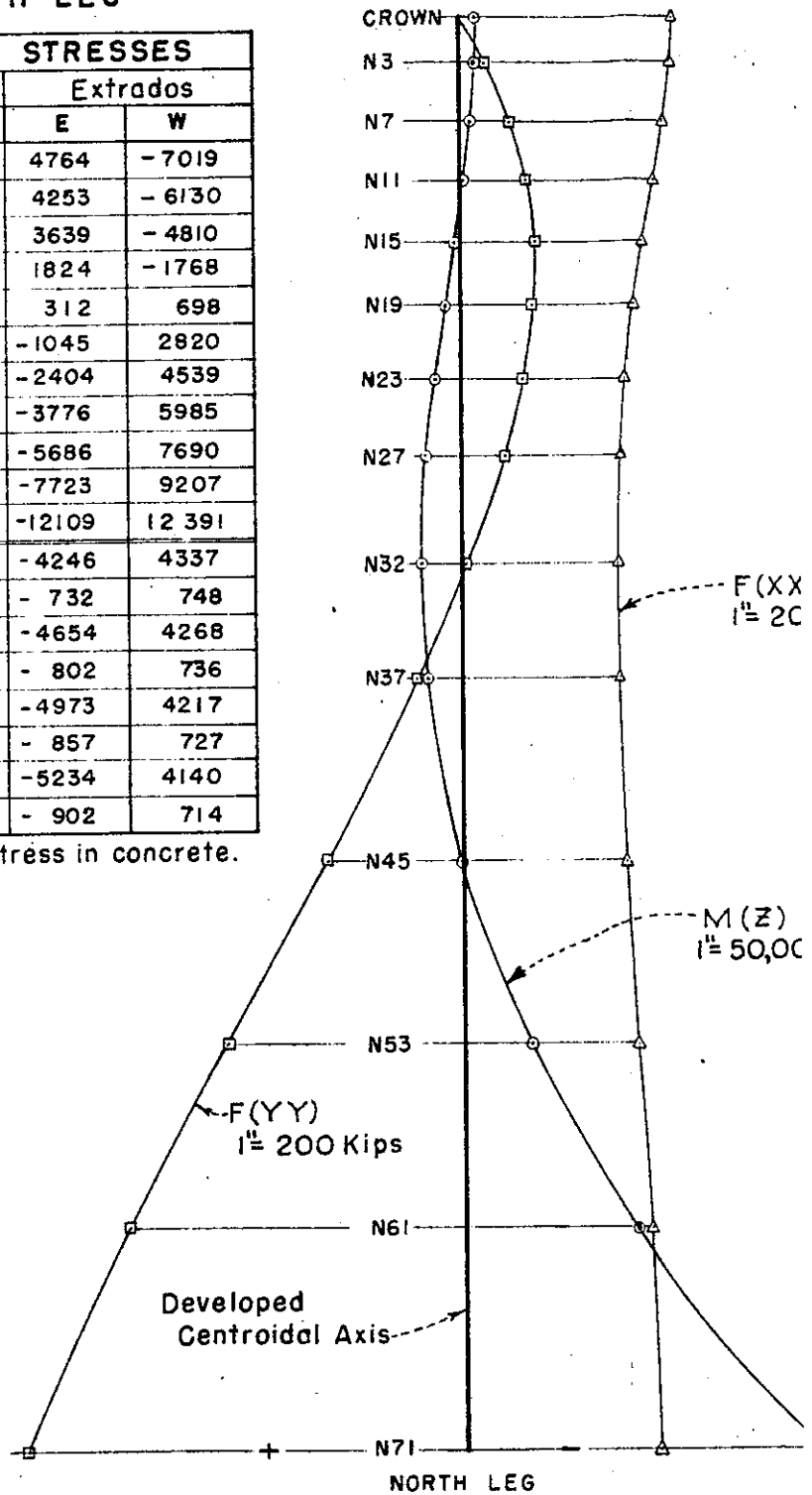
NORTH LEG

NORMAL STRESSES			
Arch Sta.	Intrados	Extrados	
	S	E	W
CROWN	3445	4764	-7019
N3	3040	4253	-6130
N7	2304	3639	-4810
N11	866	1824	-1768
N15	-262	312	698
N19	-1107	-1045	2820
N23	-1577	-2404	4539
N27	-1735	-3776	5985
N32	-1587	-5686	7690
N37	-1110	-7723	9207
N45	56	-12109	12391
N45	16	-4246	4337
* N45 C	3	-732	748
N53	464	-4654	4268
* N53 C	80	-802	736
N61	818	-4973	4217
* N61 C	141	-857	727
N71	1142	-5234	4140
* N71 C	197	-902	714

* Indicates stress in concrete.



ARCH DISPLACEMENTS

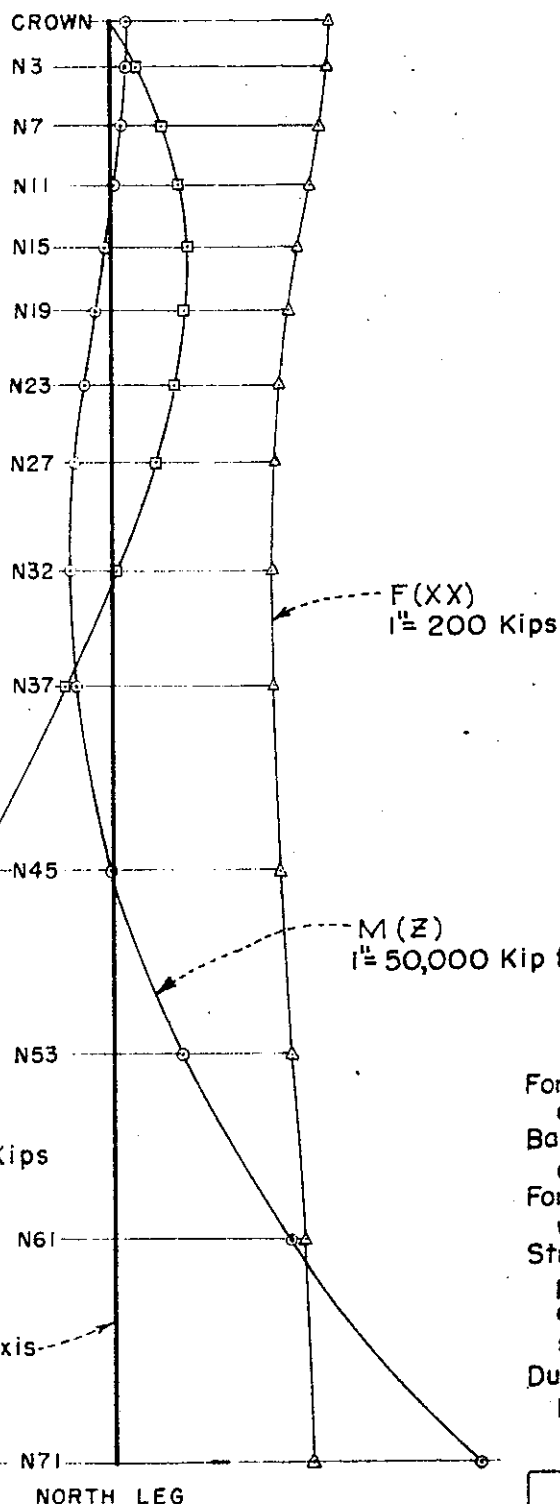


ARCH MOMENTS AND FORCES
 $F(XX)$ $F(YY)$ $M(Z)$

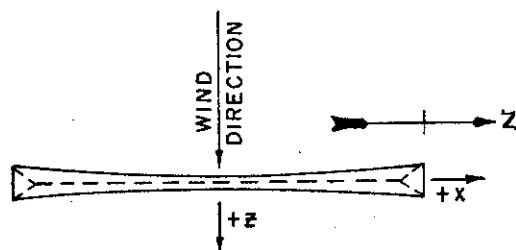
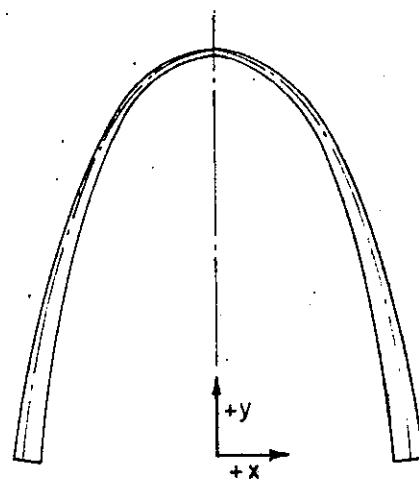
RTH LEG

L STRESSES		
Extrados		
OS	E	W
5	4764	-7019
0	4253	-6130
4	3639	-4810
3	1824	-1768
2	312	698
7	-1045	2820
7	-2404	4539
5	-3776	5985
7	-5686	7690
0	-7723	9207
6	-12109	12391
6	-4246	4337
3	-732	748
4	-4654	4268
0	-802	736
8	-4973	4217
1	-857	727
2	-5234	4140
7	-902	714

stress in concrete.



ARCH MOMENTS AND FORCES
F(XX) F(Y Y) M(Z)



NOTES

For sign conventions and notations, see drawing No. X-OA-D-899, Basic wind velocity - 75mph at 30 feet above ground.
For external force components due to west wind, see drawing No. X-OA-D-901. Stresses shown are those in exterior steel plate except locations marked with an asterisk (*) which indicate concrete stresses.
Due to symmetry, results shown for north leg only.



ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

JNEM - GATEWAY ARCH
MOMENTS, FORCES, STRESSES AND
DISPLACEMENTS - WEST WIND

DRAWN GHH SUBMITTED *Edwin Rose*
TRACED RECOMMENDED
CHECKED HWA APPROVED

DENVER, COLO., NOV. 20, 1964 X-OA-D-911

EXTERIOR PLATE STRESSES - EXTRE

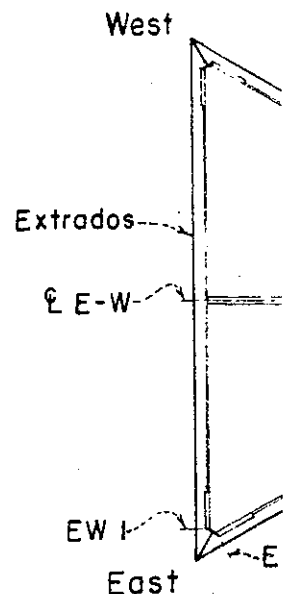
Station	* Dead Load + Wind		LOCATION NE I					LOCATION EW I				
			Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses	
	Intrados North Corner	Extrados East Corner		Temp.	Total	Temp.	Total		Temp.	Total	Temp.	Total
S 7	+ 7100	- 6800	A1	+15700	+16700	+19400	+19800	A2	-16400	- 21500	-10500	- 6400
S 15	+12000	- 7200	B1	+10500	+21000	+12000	+ 5300	A2	-16400	- 22600	- 9700	-10400
S 19	+12900	- 7700	B1	+10500	+22000	+13000	+ 4700	A2	-16400	- 23100	-10300	-11600
S 27	+10600	- 8200	A1	+15700	+21600	+ 21000	+18100	A2	-16400	- 23300	-10600	-11300
S 37	+ 7500	-12400	A1	+15700	+21400	+ 22400	+20800	B2	-14000	- 25900	- 9100	-12000
S 44	+10600	-16400	A1	+15500	+21400	+ 21500	+20900	B2	-13900	-29100	- 9000	-10500

EXTERIOR PLATE STRESSES - NORM.

Station	* Dead Load+ Wind		LOCATION NE I						LOCATION EW I					
			Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses			
	Intrados North Corner	Extrados East Corner		Temp.	Total	Temp.	Total		Temp.	Total	Temp.	Total		
S 7	+ 3500	- 4200	C1	+ 10500	+ 13400	+ 12900	+ 12100	C2	- 14000	- 18100	- 8100	- 5000		
S15	+ 7400	- 5000	C1	+ 10500	+ 16900	+ 12000	+ 8600	C2	- 14000	- 19200	- 7500	- 7700		
S19	+ 8500	- 5600	C1	+ 10500	+ 18100	+ 13000	+ 8600	C2	- 14000	- 19900	- 7900	- 8700		
S27	+ 8000	- 6100	C1	+ 10500	+ 17800	+ 14000	+ 8300	C2	- 14000	- 20100	- 8100	- 8500		
S37	+ 6700	- 8100	C1	+ 10500	+ 16700	+ 14900	+ 13100	C2	- 14000	- 22100	- 8400	- 9700		
S44	+ 8100	- 10000	C1	+ 10400	+ 17600	+ 14300	+ 13500	C2	- 13900	- 23600	- 8400	- 9100		

DEAD LOAD*			LIVE LOAD	
Sta.	Intrados	Extrados	Intrados	Extrados
Crown	-1100	-1100	+1140	- 600
3	- 800	-1300	+ 960	- 560
7	+ 600	-2000	- 790	- 380
11	+2400	-2700	-1240	+ 400
15	+3700	-3200	-1280	+ 440
19	+5000	-3900	-1180	+ 400
23	+5700	-4200	- 880	+ 280
27	+6000	-4400	- 560	+ 140
32	+6200	-4600	- 300	+ 60
37	+6000	-4700	- 160	- 160
44	+6100	-5000	+ 500	- 340

* Dead Load Includes effect of crown closure force of 642.5 kips.



TYPICAL

OR PLATE STRESSES - EXTREME LOADING CONDITIONS

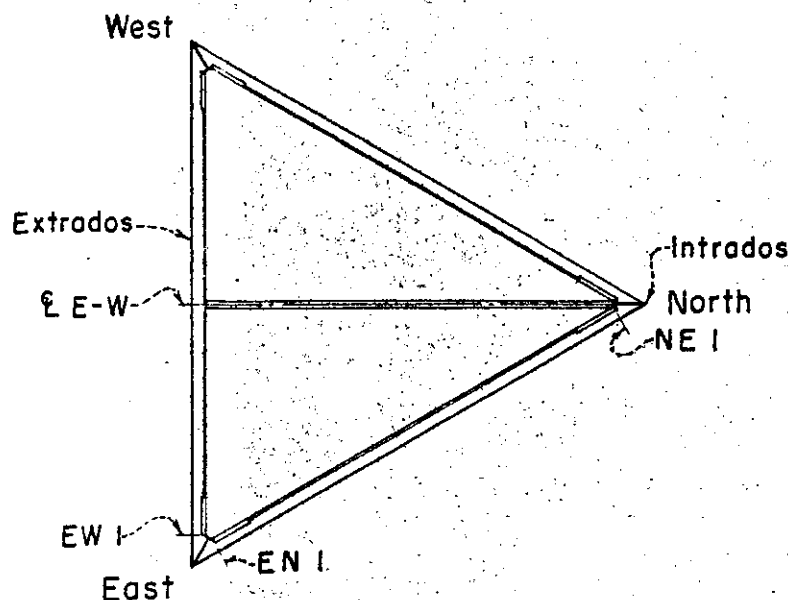
LOCATION NE I			LOCATION EW I				LOCATION EN I				L			
Max. Longitudinal Arch Stresses	Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses		Load Conditions	Max. Arch Stresses
	Total	Temp.		Temp.	Total	Temp.	Total		Temp.	Total	Temp.	Total		
3700	+19400	+19800	A2	-16400	-21500	-10500	-6400	A1	+15700	+14500	+13600	+13200	A2	-11100
1000	+12000	+5300	A2	-16400	-22600	-9700	-10400	A2	-8800	-13800	-9000	-12200	A2	-11100
2000	+13000	+4700	A2	-16400	-23100	-10300	-11600	A2	-8800	-14400	-9600	-13300	A2	-11100
1600	+21000	+18100	A2	-16400	-23300	-10600	-11300	A2	-8800	-15300	-10000	-13400	A2	-11100
1400	+22400	+20800	B2	-14000	-25900	-9100	-12000	B2	-7000	-18900	-8500	-14800	A2	-11100
1400	+21500	+20900	B2	-13900	-29100	-9000	-10500	B2	-7000	-22200	-8200	-11100	A2	-11100

R PLATE STRESSES - NORMAL LOADING CONDITION

LOCATION NE I			LOCATION EW I					LOCATION EN I					LOCATION EL I	
Max. Longitudinal Arch Stresses	Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses
	Total	Temp.		Temp.	Total	Temp.	Total		Temp.	Total	Temp.	Total		
3400	+12900	+12100	C2	-14000	-18100	-8100	-5000	C1	+10500	+10400	+9100	+7600	C2	-10500
3900	+12000	+8600	C2	-14000	-19200	-7500	-7700	C2	-6100	-10400	-6900	-9300	C2	-10500
3100	+13000	+8600	C2	-14000	-19900	-7900	-8700	C2	-6100	-10900	-7200	-10200	C2	-10500
7800	+14000	+8300	C2	-14000	-20100	-8100	-8500	C2	-6100	-11700	-7500	-10300	C2	-10500
3700	+14900	+13100	C2	-14000	-22100	-8400	-9700	C2	-6100	-13800	-7800	-11300	C2	-10500
7600	+14300	+13500	C2	-13900	-23600	-8400	-9100	C2	-6100	-15300	-7500	-9100	C2	-10500

* LIVE LOAD		
ados	Intrados	Extrados
00	+1140	-600
100	+960	-560
200	-790	-380
300	-1240	+400
400	-1280	+440
500	-1180	+400
600	-880	+280
700	-560	+140
800	-300	+60
900	-160	-160
1000	+500	-340

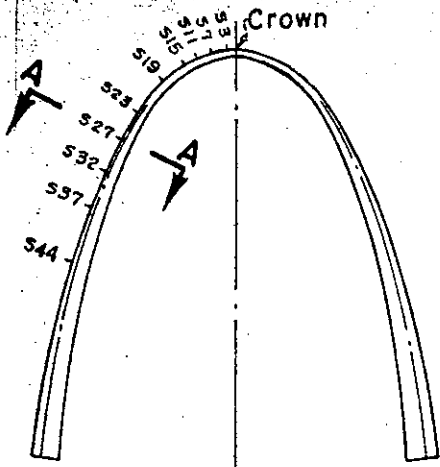
ides effect of crown
of 642.5 kips.



TYPICAL SECTION A-A

LOADING CONDITIONS

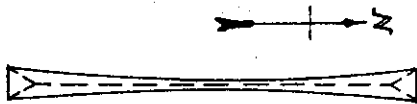
LOCATION EN I				LOCATION E-W				
Longitudinal Stresses		Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses	
St.	Total	Temp.	Total		Temp.	Total	Temp.	Total
0	+14500	+13600	+13200	A2	-11700	-16200	-15000	-18500
0	-13800	-9000	-12200	A2	-11700	-17800	-14900	-19700
0	-14400	-9600	-13300	A2	-11700	-18400	-14500	-19100
0	-15300	-10000	-13400	A2	-11700	-17800	-13100	-16600
0	-18900	-8500	-14800	A2	-11700	-17300	-13100	-18000
0	-22200	-8200	-11100	A2	-11800	-18600	-12000	-13800



ELEVATION

LOADING CONDITIONS

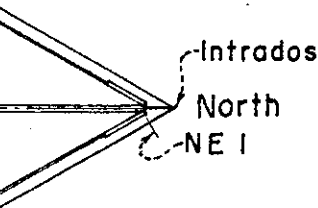
LOCATION EN I				LOCATION E-W				
Longitudinal Stresses		Accompanying Trans. Stresses		Load Conditions	Max. Longitudinal Arch Stresses		Accompanying Trans. Stresses	
St.	Total	Temp.	Total		Temp.	Total	Temp.	Total
0	+10400	+9100	+ 7600	C2	- 10000	-13700	- 12900	- 16000
0	- 10400	- 6900	- 9300	C2	- 10000	-15200	- 12900	- 17200
0	-10900	- 7200	-10200	C2	- 10000	- 15800	- 12600	- 16700
0	-11700	- 7500	-10300	C2	- 10000	-15500	- 11400	- 14900
0	-13800	- 7800	-11300	C2	- 10000	-15300	- 11400	- 15700
0	-15300	- 7500	- 9100	C2	- 10000	-16300	- 10300	- 11900



PLAN

NOTES

Stress analysis based on arch design as shown on Eero Saarinen & Assoc. Drawing No. AR-1 (revised 9-20-63) and Severud-Elstad-Krueger Assoc. Drawing No. S-101 thru S-120 (including revisions of 4-4-64 on Drawing No. S-113, S-114 and S-115).
See drawing X-OA-D-899 for notations and sign conventions. + = tension
- = compression.
All stresses are in psi.
All stresses are longitudinal arch stresses unless otherwise noted.



N A-A

⊕ ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	
JNEM GATEWAY ARCH	
MAXIMUM LONGITUDINAL ARCH STRESSES ABOVE STATION 45	
DRAWN GHH	SUBMITTED <i>Edwin Rose</i>
TRACED	RECOMMENDED
CHECKED HWA	APPROVED
DENVER, COLO., NOV. 20, 1964	
X-OA-D-912	

Chapter V

ARCH--DETAIL ANALYSIS

1. Stations 0 to 45

(a) Stability of Sides of Arch Rib

A study of available literature did not yield any specific solutions of anything similar to the problem of buckling of the sides of the arch rib. From the usual assumptions for orthotropic plate design 11/, it appeared that the stiffness in the transverse direction would be of the same magnitude as in the longitudinal direction and, therefore, the section would be quite stable. We decided that the best procedure would be to build, of plastic material, a model of the arch rib side and determine the stiffness in both directions as well as the torsional stiffness. The general procedure for this is outlined on page 45 of "Theory of Plates and Shells" by S. Timoshenko and S. Woinowski-Krieger 9/. We also decided to test the model to determine critical load from which a comparison with theoretical calculations could be made, and also the results would be converted to critical buckling stresses in the prototype. The procedure for determining critical load experimentally is outlined on page 177 of "Theory of Elastic Stability" by S. Timoshenko 10/.

Two models were constructed because local weaknesses in the first model led to early failure when loaded for critical load. (See Appendix E for report of tests.) The second model was then constructed so as to eliminate the faulty details. Very satisfactory results for critical load were then obtained. However, the various stiffnesses, discussed above, were determined from both models and these were in good agreement. The elastic constants for the prototype, as determined experimentally are as follows:

$$\frac{E_x I_x}{(1-\nu^2)} = \frac{3.47 \times 10^8}{(\text{longitudinal})} \quad \frac{E_y I_y}{(1-\nu^2)} = \frac{3.04 \times 10^8}{(\text{transverse})} \quad G_{xy} I_{xy} = \frac{1.13 \times 10^8}{(\text{torsional})}$$

The above values are in units of pounds inch² per inch width or pounds inch.

It appears that the transverse stiffeners, which are the legs of the triangular transverse frames at arch stations, are sufficient to develop nearly the full width of skin plates between transverse frames. This is quite consistent with studies made in orthotropic plate design, which has led to the assumption that the effective plate width is one-third of the simple span length for an orthotropic plate subject to transverse load. See page 43, "Design Manual for Orthotropic Steel Plate Deck Bridges" published by AISC 11/.

Since the moment of inertia of the sections with two skin plates consists largely of the skin plates and only a small amount is contributed by the web plates between them, it is reasonable to expect the stiffness in both directions to be nearly equal. In other words, so far as the stability of the side plates is concerned, the longitudinal diaphragms are excessive in number; considerably fewer could be used and the skin plates would still act together. However, the diaphragms are required to prevent local buckling, as is discussed elsewhere.

The models were constructed of Plexiglas to a scale 0.12 to 1.0 of prototype. Figure V-1 shows the first model. Both models were made uniformly straight with properties as given for Station 37 (original design). The total length was three panels long which produced a member approximately square--51 inches long and 48 inches wide. The top and bottom ends only were loaded and in such manner as to provide only simple support of these edges. In the first model, the sides were simply supported the full length, but in the second model the sides were simply supported only at the points where transverse diaphragms occur. This was done because by the time the second model was built, it appeared we would recommend pin connections in the transverse triangular frames. The point support used is on the safe side because in the prototype a certain amount of support is provided along the entire outside edges of the face by the continuity of outside skin plates.

It was mentioned earlier that the first model failed because of certain weaknesses in details. This refers to the design details that call for longitudinal diaphragms and stiffeners to be made discontinuous at the transverse frames. At the ends of the diaphragms and stiffeners, high stress concentrations appeared almost immediately in the skin plates, as shown in the top photograph of Figure V-2. This is to be expected since there is an abrupt change of section. As the load increased, the diaphragms separated from the outer skin plate and failure of this skin plate followed very shortly. This is shown in the lower photograph of the figure. As this failure occurred at such a small load, it was impossible to determine the critical load.

The second model was constructed to correct these weaknesses by making the longitudinal diaphragms and stiffeners continuous. This model produced good results, as shown in Figure V-3, and remained intact. The critical buckling load for prototype when converted to unit stress and using the tangent modulus as given for carbon steel on page 343 of "Buckling Strength for Metal Structures" by Bleich 12 is $\sigma_{cr} = 31,500$ psi.

Tests were conducted in the laboratory (see Appendix D) to determine the proportional and elastic limits of stainless steel. Neither of these is well defined. The elastic limit as defined by the 0.2 percent tangent offset method is equal to or greater than that for A7 steel. However, there is no apparent proportional limit but the stress-strain relation is a curved line for its entire length. This fact makes it difficult to predict the true buckling strength of the section. A study would require more time than is available. Therefore, we could only assume the properties of stainless steel to be the same as A7 steel.

We conclude that the sides of the arch rib, acting as a unit, are almost as stable as can be constructed provided the longitudinal diaphragms and stiffener angles are made continuous.

The failure of the first model and the improvement accomplished by making the longitudinal diaphragms and stiffeners continuous, raised the question as to the true effect of stopping the diaphragms and stiffeners short of the transverse triangular frames. It was apparent that the critical point was in the vicinity of the ends of the diaphragms where failure of the first model began. Accordingly, tests were made in the photoelastic laboratory to determine the nature of the stress distribution at this point. These tests are described in Appendix E.

The results for a 1-inch gap between ends of diaphragms showed a stress concentration at the point with one principal stress σ_2 equal to 3.38 times average stress over the section and the accompanying principal stress σ_1 equal to 0.4 times the average stress but of opposite sign. From the standpoint of the more accepted theories of failure, this means that failure occurs when the larger principal stress reaches approximately 0.93 yield point. If transverse stresses are present in the sides of the arch, and they do exist in the vicinity of the transverse triangular frames, these would tend to increase the ratio of σ_1 to σ_2 and failure would then occur at an even smaller stress.

On the basis that $\sigma_2 = -8.20 \sigma_1 = 3.28$ average stress, failure occurs in the outer skin plate where the average stress $\frac{0.93 \sigma_y}{3.28} = 0.28 \sigma_y$.

Here again, the use of stainless steel presents a problem because the yield point is so vaguely defined. If we use $\sigma_y = 33,000$ psi, the same as A7 steel, then the average stress on the member at which the outer skin plate begins to fail in the vicinity of the end of the diaphragm $= 0.28 (33,000) = 9,240$ psi. However, this area is quite extensive. Furthermore, this is the precise location of the end spot weld that connects the diaphragm to the outside skin plate so that

any additional stress from the action of the diaphragm such as shear will bring about failure of the end spot weld. This does not relieve the situation, but the point of concentration moves to the next spot weld where the same previously explained conditions take place again leading to failure at the next spot weld; and the process continues until failure of the side as a whole takes place.

If the diaphragms are not made continuous, the average stress should be limited to $9,240 \div$ safety factor which obviously limits the value too much since a large safety factor must be used because of transverse forces present. Therefore, we maintain that it is absolutely essential the diaphragms and stiffeners be made continuous as was recommended in the second interim progress report.

It will be noted that tests were made for a 4-inch gap between ends of skin plates, corresponding to the original design. This gap gave less concentration of stress, $\sigma_2 = 2.18$ average stress and $\sigma_1 = 1.20$ average which would raise the average stress at failure to 11,100 psi. This is too low, also.

The prime function of the longitudinal diaphragms is to make inner and outer skin plates work together--a very important function. However, the diaphragms will develop stress equal to the adjacent skin plates within a very short distance of their ends, approximately 4 inches. This must take place unless a series of very short diaphragms or some form of lacing is used. Neither of these methods could be recommended; they would be more expensive and less satisfactory.

Assuming then that the present design is used, except that diaphragms be made continuous, there is a certain amount of shear that must be developed between diaphragms and skin plates. To do this, a new spot weld technique has been used. Fillet welds were not used to connect the 1/4-inch outside plates because of distortion and the resulting undesirable appearance on the outside.

Test data presented to us showed the above spot welds to be good for 7,000 pounds each. A continuing test program is reportedly being followed to assure that the quality of these welds is maintained. Our computations on the basis of continuous diaphragms show that the welds should then be adequate. However, we are reluctant to accept these tests as conclusive for the reasons discussed in Chapter II.

(b) Thermal Stresses

In the present design, utilizing rigid corner connections in the transverse triangular frames, high thermal stresses will develop in exterior and interior skin plates in both longitudinal and transverse directions. These stresses are due to the large difference

in the coefficients of linear expansion between the stainless steel (9.6×10^{-6} per $^{\circ}\text{F}$) and the carbon steel (6.5×10^{-6} per $^{\circ}\text{F}$), and the temperature differences which will exist between the interior and the exterior plates.

An analysis was made of thermal stresses in the skin plates in both the longitudinal direction (parallel to the arch centroidal axis) and the transverse direction. Details of this analysis are given in Appendix B. The analysis of stresses in the longitudinal direction shows a maximum value of 11,800 psi compression in the outer skin at the center of the sides and 16,300 psi near the corners. These stresses can extend over considerable area and must be considered primary stresses.

Transverse stresses depend on the action of the transverse triangular frames which are described in Part (c) of this chapter.

Assuming modified rigid corners in the frame, compressive stresses develop in the transverse direction as high as 15,200 psi in the exterior skin plate at Station 15. These transverse stresses must be combined with the high longitudinal compressive stresses which greatly affect critical local buckling problems in the skin plates. These effects are summarized on Sheets Vf 23 and 24.

This condition can be relieved and the transverse thermal stress virtually eliminated by introducing a hinged connection at the corners of the transverse triangular frames. Stresses in the longitudinal direction will develop as indicated, but use of the hinge will prevent further stress buildup due to lateral loads.

(c) Transverse Triangular Frames

Some changes have been made in design assumptions from those used when preparing the Second Interim Progress Report, June 11, 1964. Of greatest significance is the addition of a support to the center of the extrados leg of the frames. Although a support was shown on Structural Engineer's Drawing, Sheet S118, the connections for it were considered inadequate to perform the intended function and therefore the support was neglected. An improved support had been added and detailed on the contractor's Drawing No. GA 100, Sheet 2 of 2.

This present report also goes into more detailed calculations based on the assumption of rigid corners at the transverse triangular frames. The work of the interim report was only partially carried out to the point where it was evident that weaknesses were inherent in the rigid corners.

The present report has carried out investigations for both rigid and hinged corners so that a direct comparison can be made. This is done only for the frames at Stations 15 and 44 since time does not permit a complete analysis.

Some modifications have also been made regarding temperatures of inner and outer skin plates. These are carefully explained in Appendix B. Some changes were also made in the combinations of temperature and wind assumed for design. A summary of design load combinations is shown on Sheets Vf 3 and 4. These combinations are discussed in Chapter IV.

The present design attempts to effect a rigid connection at the corners between the individual legs of the triangle. This produces a triangular-shaped rigid frame. However, it is very difficult to predict how much stiffness the corners and legs of the frame actually possess. This is due to uncertainties which center around the effectiveness of the skin plates. In other words, the widths of skin plates that effectively work with the legs of the transverse frame will vary. Furthermore, it would be very difficult to determine this variation.

In orthotropic plate design 11/, it is accepted practice to assume the effective plate width equal to 0.333 of the span length, where span length refers to the distance between points of contraflexure. In the region of negative moments, this rule no longer applies. It can only be said that for a section close to the point of contraflexure, a much smaller width of plate will be effective. A theoretical approach to the problem is developed on page 171, of reference 19/. This is for isolated beams. The analysis has been extended to multiple beams. See reference 23/, page 603. The nature of the problem is described as shear lag. Due to shear deformations, the stress varies across the width of the plate and could be described as lagging as the distance from the web increases; consequently the plate is less effective at greater distances from the web.

The theoretical approach arrives at effective plate width equals 0.363 span length for simple spans 19/, with moment curve of sine wave configuration. For continuous spans, with equal moments over the support and at the center of span, the effective width equals 0.181 span length for a cosine curve and 0.154 for straight line distribution, reference 19/, 23/. Comparing the sine and cosine curves above, the rule may be stated as 0.363 span length, if we let span length equal the distance between points of contraflexure. The straight line moment curve would produce effective widths of slightly less value or 0.308. It can be seen that the orthotropic design rule 11/ is in close agreement using a value, 0.333, which is a good average of the above values and may represent a more general case of loading which would produce a moment curve that is a combination of sine, cosine, and straight line moment curves. It should be emphasized that this effective plate width has been worked out only for determining stresses; it does not tell us how much is effective for determining rigidity.

Neither of the above methods gives answers for cases where the bending moment diagram represents conditions that exist in the triangular frames and in particular what the effective plate width is at the corners of the frames. There are two important differences: First, the theoretical approach 19/, 23/ and orthotropic design 11/ are based on continuous fixed beams that do not make an abrupt change in direction as the frame does. Second, in the actual problem, moments at the supports (corners) are not equal to moments at the center of span and the distance from support to point of contraflexure is not one-fourth of the span as assumed in the above theoretical approach. We know, and this will be verified later, that the stress in the outer skin plate will be zero at the apex of the corner, which is equivalent to saying none of the outside skin plate as it affects stiffness is effective at this point.

In studies of corners in rigid frames 13/, 14/, 15/, tests show that stress does not travel from point to point around a sharp corner such as path a-a, Figure V-4, but rather tends to take a more direct smooth path, such as b-b, and that metal placed at the most remote portion of a rigid frame corner contributes practically nothing to the effective metal at the joint. In the summary on page 870, reference 14/, it is stated: "The stresses at the outer corner of the knee specimen were small, and reinforcing the corner had a negligible effect on the distribution of stress in the specimen." This would be even more pronounced in the acute, 60° joints of the transverse triangular frames which make our assumptions as to effective plate width more uncertain because we must assume that at the apex of the corner the outer skin plates are ineffective. Their effectiveness would increase some as the distance from the apex increases, but then must be limited again as we approach the point of contraflexure.

Near the corners at the ends of the legs of the frame of the present design, Section A-A, Figure V-4, the 1-3/4-inch inside skin plate is, for all practical purposes, not connected to the corner. A 1/2-inch continuous diagonal plate P1 is welded across the corner to the 1-3/4-inch skin plates, but this plate connects almost perpendicular to the skin plate and cannot induce any appreciable direct stress into the skin plate.

A study of Figure V-4 indicates what will take place when a corner moment is applied as shown. Any stress in P1 must transmit a shear force to the web of the 12 WF piece. Plate P1 will pull almost at right angles to the 1-3/4-inch skin plate which in turn is not directly connected to the web but is held by bolts through the flange of the 12 WF. The flanges will deflect under this pull as shown in Section A-A, thereby permitting the 1-3/4-inch plate to pull away

from the 12 W^F piece as shown. Stress will not travel this indirect, flexible path but will go the direct rigid path through the encircled portion of the corner. This portion is quite rigid up to the point where welds fail. Also, when considering Section A-A, the 1-3/4-inch skin plate cannot be assumed to act with the rest of the section (the piece of 12W^F36 and the 1/4-inch skin plate) because it is not connected in any manner to them beyond this point toward the corner. From all these considerations neither the 1-3/4-inch skin plate can be considered effective at Section A-A, nor the plate Pl at the corner.

Similar reasoning will rule out both of these plates when the applied moment is reversed because it is more than likely that the 1-3/4-inch skin plate cannot be drawn up tight against the flange of the piece of 12W^F36 and a gap here would render these pieces even less effective. Since the analysis of the corner is so vital to the assumptions made in the structural behavior of the entire arch, it was decided to make model tests of the corner. These are described in Appendix E. The results showed agreement with previous tests 13/, 14/, 15/ along Section A-A, Figure V-4, the only area in which they are comparable.

As a result of the above tests, the critical sections and stress distributions were assumed as shown in Figure V-5. A comparison of calculated and measured stresses along Section A-A are shown in Figure V-6. It should be noted that measured values exceed calculated values at some points and therefore the assumptions are not always on the safe side although they are in reasonably good agreement.

It should be emphasized that no attempt was made to carry the analysis beyond the point where stresses exceed the yield point, or into the plastic range, because as stated before we are not advocating a plastic concept of this design.

In the above calculations, except for the recommended hinged corner design, the extrados side of the triangular frame has been assumed to be supported rigidly at the centerline by the strut. This can be accomplished quite well if many precautions are observed, but it is our opinion that these are impractical. The primary difficulty centers on thermal effects. If the triangular unit of the arch is assembled in the yard at one temperature then erected and the strut that supports the extrados side is connected later on at a different temperature, the combined effect would be the same as a lengthening or shortening of the strut and could partly nullify any benefits of the support for the extrados side or possibly have a detrimental effect. If the strut were to have as much as 1/8-inch effective lengthening, it would modify the moment for the frame at Station 15 as shown on Sheet V^f 7. It is difficult to control or

predict the above effect; therefore, no specific recommendations are to be made concerning this. However, this will be kept in mind later when we consider safety factors.

It should be pointed out at this point that every assumption made so far regarding the frames has, in our opinion, favored the present design, such as effective plate widths and the efficiency of difficult-to-make welds, and therefore higher safety factors must be maintained.

As stated at the beginning of this chapter of the report, calculations for the interim report did not assume the extrados support and in fact was not necessary for the hinged frames proposed. However, the support would benefit some parts of the hinge design, and it is possible that the recommendation of the interim report to increase the distance between skin plates between Stations 27 and 45 may not be necessary if the extrados support is considered. It is our opinion that a better all-around design is achieved by using hinges and omitting the struts as assumed in the second interim report. Specifically, this is to increase the distance out to out of skin plates, uniformly from 14-9/16 inches at Station 45 to 9 inches at Station 36, then to 7-3/8 inches at Station 27. Therefore the analysis was made at Stations 15 and 44 using hinged joints without the strut.

Transverse triangular frames at Stations 7, 15, 19, 27, 37, and 44 were analyzed by the general purpose indeterminate structural analysis program. In addition, special purpose computer programs were developed to perform the following tasks:

- (1) Compute loads on triangular frames due to components of longitudinal arch stresses
- (2) Compute parabolic stress distribution across corners of frames
- (3) Compute transverse stresses in exterior and interior plates
- (4) Determine maximum longitudinal stresses and associated transverse stresses, and the maximum transverse stresses and their associated longitudinal stresses, for various load combinations
- (5) Compute the minimum factors of safety against stress failure

For more complete information on development of computer programs, see Appendix F.

A check of the stresses in transverse triangular frames was obtained by independent analysis using desk calculator methods.

Computations for the transverse frame at Station 15 are carried out in detail on the following pages, Vf 1 to 33, inclusive, to provide a guide for anyone wishing to follow the analysis.

In computations that follow, the frames at Stations 15 and 44 are analyzed for various loading conditions, also for the frames as detailed, with modified joints and with hinged joints. Two corner modifications are considered at Station 15. Modification No. 1 was tried because it would require a minimum amount of change, but in our opinion the resulting corner is not adequate; therefore, Modification No. 2 was designed. Only one modification of the corner at Station 44 was considered in the computations which is similar to Modification No. 2 for Station 15. These computations show that it will be unsafe to use any type of rigid corner in the vicinity of Station 44. Modification No. 2 for Station 15 is detailed on Drawing No. X-OA-D-908. Hinged joints are detailed on Drawings No. X-OA-D-903 and X-OA-D-905.

The tables shown on sheets Vf 11 to 14 summarize the transverse stresses in the frames at various sections along the arch for the different loading conditions. These tables do not show safety factors because longitudinal stresses must also be considered. However, it is evident the transverse stresses themselves are excessive.

The rigid corners, as presently designed, will fail first in the 1/4-inch fillet welds along Section A-A, Figure V-5. See results pages Vf 8 and 9. These welds could be strengthened but the base metal in this area is overstressed. The principal stresses along Section A-A are of nearly equal magnitude but opposite sign which is probably the worst possible combination. The various theories of failure for ductile metals show that failure must occur in the web plate in this area. The condition created here is not one in which the stress is always of the same sign but reverses as shown for Case A1 and A2, Section A-A South, Sheet Vf 11. Reversal beyond yield point will ultimately produce cracking or total failure.

In the present design the 1/4-inch outside skin plates would be stressed in compression in two directions at several points around the frame so as to bring about buckling failure. Buckling is not necessarily always critical, but if it is known to exist a careful study must be made to determine how this affects other design assumptions and this would involve more study than can be justified when considering the relative costs of engineering and increased material. Since this can easily be avoided, it is far better and more economical to avoid any buckling.

Modification No. 1 of the frame at Station 15 improved the stress conditions very much at Section C-C, Sheet Vf 2 as well as Section D-D.

In fact, in a temporary type of structure this detail might be considered by some to be adequate. However, the analysis of the corner does not take into account all the secondary effects of shear and bending in the various elements of the corner or locked-in stresses that are present in such a heavy weldment. This fact plus the knowledge that the welds in the corner are so difficult to make and inspect should require a much more substantial safety factor, about 2.5, in the corner itself. It certainly is folly to make the details, such as the frame corners, weaker than the component parts they connect. Furthermore, we are designing a monumental structure without precedent and the principle regarding safety factors should be to increase rather than reduce those used in ordinary structures. The minimum safety factor used in ordinary design for the most simple beam is 1.65.

The modified corner shown on Drawing No. X-OA-D-908 has a safety factor of 2.5 plus. If this detail, or something equally strong, had been incorporated in the original design, these modifications would have added very little to the cost of the structure.

It is very clear that many advantages are derived from the hinge. The analysis is made simpler and uncertainties in assumptions are greatly reduced. Less metal will be required, difficult welding will be eliminated, and the resulting structure is permitted to "breathe" with temperature variations, thereby almost eliminating thermal stresses in the transverse direction. Buckling conditions are also greatly improved.

It may be well to enlarge upon the above statement about use of the hinge, that it is simpler and eliminates uncertainties. The hinges create sides on the transverse frames that are no more complicated than simple span beams. Here moments, thrusts, and shears are statically determinate; they are not at all dependent on the stiffness of the sides, and as far as effective width of skin plates are concerned we are only interested in how much is effective for determining stresses in a simple span. For this we have answers that are well established.

The rigid corners create transverse triangular frames that are six times or more statically indeterminate. The moments, thrusts, and shears created by transverse loads and thermal forces are dependent on the stiffness of the sides. This varies continuously along all three sides of the frame, and even varies for the different loadings. We have no criteria to determine the effective width of skin plate acting to give us the stiffness of the sides under the simplest conditions. It has been pointed out that the only data available for effective skin plate widths apply only to stress distribution. It is in some ways a misnomer to call it effective width because

the stress is not cut off completely in the region beyond this width and we cannot say, therefore, that this so called effective width is all that affects the stiffness of the frame. Our analysis has been based, however, on just such an assumption because we have no better. However, we made it consistent throughout. That is, whatever plate width was assumed for stiffness was also used for determining stress. If we were to increase effective plate widths over those assumed and again keep our assumptions consistent, we can quickly reach a point where the increased width begins to hurt more than help. The critical point of stress may shift from outside to inside skin plate and from the center of the span to the corners or vice versa. It may even be that our assumed stiffnesses are not enough in certain areas as regards frame action which means that actual moments are greater than assumed. At the same time, the effective plate widths as regards stress may be overly generous. The combination of increased moments and less effective plate width could greatly increase stresses over those assumed.

The hinge design is free of all these uncertainties even in the very corners themselves. In the computations that follow, it can be seen that assumptions made at the corners are entirely on the safe side not only at the corners but as they may affect the entire sides of the transverse triangular frames.

The frame with the hinges is still capable of transferring shear at the corners through the bent plate and the outer skin plate. This was demonstrated by tests in our laboratory. It should be emphasized that for the bent plate to be effective it must be continuous throughout, and the ends at Station 45 be supported as shown on Drawing No. X-OA-D-906, in order to provide the necessary end shears on the plate. The continuous bent plate, supported as shown, is then essentially in pure shear and accomplishes exactly what is done by a diagonal plate across the corner. The above tests demonstrated the effectiveness and are set forth in Appendix E.

A small amount of restraint to rotation is developed by the hinge, but this is very small compared with the fixed end moments caused by thermal effects and other loads. In detailed computations, restraint from the hinges is neglected entirely, since a check showed that the influence is less than 5 percent.

Stresses introduced by the hinge, do not influence buckling of the outside skin plate or the inside bent plate. These stresses are transverse to the longitudinal direction and are predominantly flexure stresses; the direct stress is approximately 100 psi. The principle effect of flexure stresses will be to introduce a curvature to the plates, transverse to the direction of the longitudinal stresses. The nature of this curvature is such that it will not reduce the allowable buckling stresses in the longitudinal direction.

A comparison of safety factors for the frames at Stations 15 and 44 can be obtained from Sheets Vf 21, 23, 24, 29, and 31 to 45, inclusive. In these tables the combined effects of longitudinal and transverse stresses are considered. Shear stresses in general are small and do not affect the results. The safety factors shown are based on the assumption that the longitudinal diaphragms and stiffeners are made continuous and that additional transverse stiffeners will be added as discussed on Sheets Vf 24 to 27. If these conditions are not fulfilled, the above safety factors are greatly affected. This is discussed in Section 1(a) of this chapter.

At Station 44 the longitudinal stresses are excessive amounting to 29,600 psi of which 13,900 is due to thermal effects. The only feasible solution to this, provided the basic design concept of cellular construction is maintained, is to change the 1/4-inch stainless outside skin plate to 1/4-inch stainless clad steel. By so doing, the thermal stresses are reduced to 8,400 psi and the total stress to 24,100 psi. This gives a safety factor = 1.26 but precludes the existence of any sizable transverse stresses which in turn rules out the use of rigid corners.

The above excessive stresses exist at Station 44 and will lessen somewhat at stations above this point, but time does not permit further investigations. Therefore, the extent that the outside skin plate must be changed from stainless to stainless clad has not been determined.

Since changing from stainless to stainless clad requires refabrication of all sections of the arch above Station 45 that have already been fabricated, we are led to consider a new concept in the present design that eliminates all the troublesome problems, particularly thermal stresses. This proposal is shown on Drawing No. X-OA-D-913. It departs from the cellular construction and, therefore, simplifies fabrication and erection very much. Using heavy stainless clad steel eliminates welding problems inherent in the present design, and the ordinary established methods of welding can be used. Stresses are greatly reduced and the entire structure above Station 45 would be open to full visual inspection. The advantages of this proposed change should reflect very much in cost of the future fabrication and erection of the arch. On Sheets Vf 46 to 48, a preliminary study at Station 37 shows that the proposed section is quite adequate. Safety factors are 2.0 or greater. The total area of steel is approximately the same as before so that very little weight change is to be expected. It should be very apparent though that fabrication and erection costs would be greatly reduced.

2. Stations 45 to 71

Since this portion of the arch has already been completed, it will be difficult to make any modifications now. The analysis has not been

carried out completely, as discussed in Chapter IV, but the present studies show that an undesirable condition now exists at Station 45 which affects the upper portion of the arch. This condition will be discussed and recommendations to correct it will be made in this chapter.

Of greatest significance is the probability that the outside skin plate will not act integrally with the concrete and the inside skin plate. Thermal effects, studied in Appendix B, show that the outside plate will be sheared free from the concrete, even when only the transverse effects are considered. This is true from Station 45 all the way down the arch rib. The plate will remain attached at the corners to the outside skin plates on the other two sides.

The inside skin plate is much more securely held to the concrete and in the center portion, according to our computation, will remain attached and act together with the concrete. The heavy inside skin plates near the corners will not act together with the concrete in the transverse direction although they remain attached. Therefore, studies from Station 45 to 71 are based on a composite section composed of concrete, some embedded steel and under certain conditions, the inside skin plates.

Transversely, the stresses are very small. At Station 46, if a 1-foot-wide transverse frame is considered, the moments for the lateral wind forces are approximately as shown in Figure V-7. These forces produce maximum stresses of 100 psi or less in the concrete. Combined stresses including shrinkage and thermal forces reach maximums of 650 psi tension in the concrete and 2,900 psi compression in the steel. The concrete stress is high but still has a safety factor of approximately 1.2.

Longitudinal stresses are discussed in Chapter IV.

A critical condition does exist at Station 45 that must be corrected. The outside skin plate above Station 45 must be counted upon to act with the inside skin plates and diaphragms and this plate is stressed to a maximum of 24,100 psi. Now immediately below Station 45, as pointed out before, the outside skin plate is not attached to the concrete so it is unsupported the full width, a distance equal to 40.75 feet. Obviously, this plate cannot support the skin plate above Station 45, to which it is made continuous by welding. Some provision must be made, therefore, to transfer the stress from the outside skin plate above Station 45 to the top of the concrete at this point. A suggested method is shown on Drawing No. X-0A-D-906.

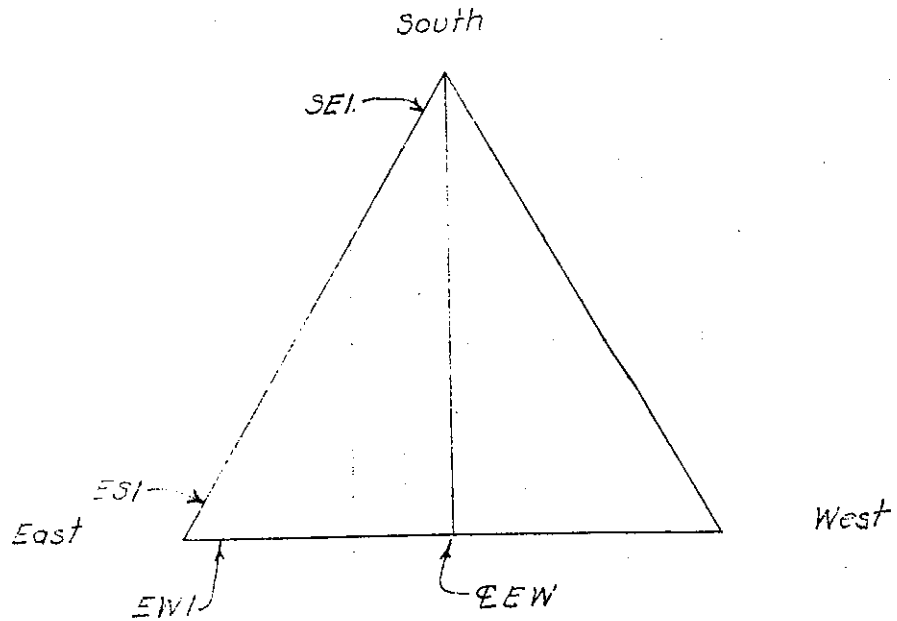
From another viewpoint, it may not be entirely satisfactory if the outside skin plate is unattached to the concrete. This condition may produce unsightly buckling of the outside plate but this is an architectural matter and we make no recommendation concerning it.

C.A.S.
W.L.L.

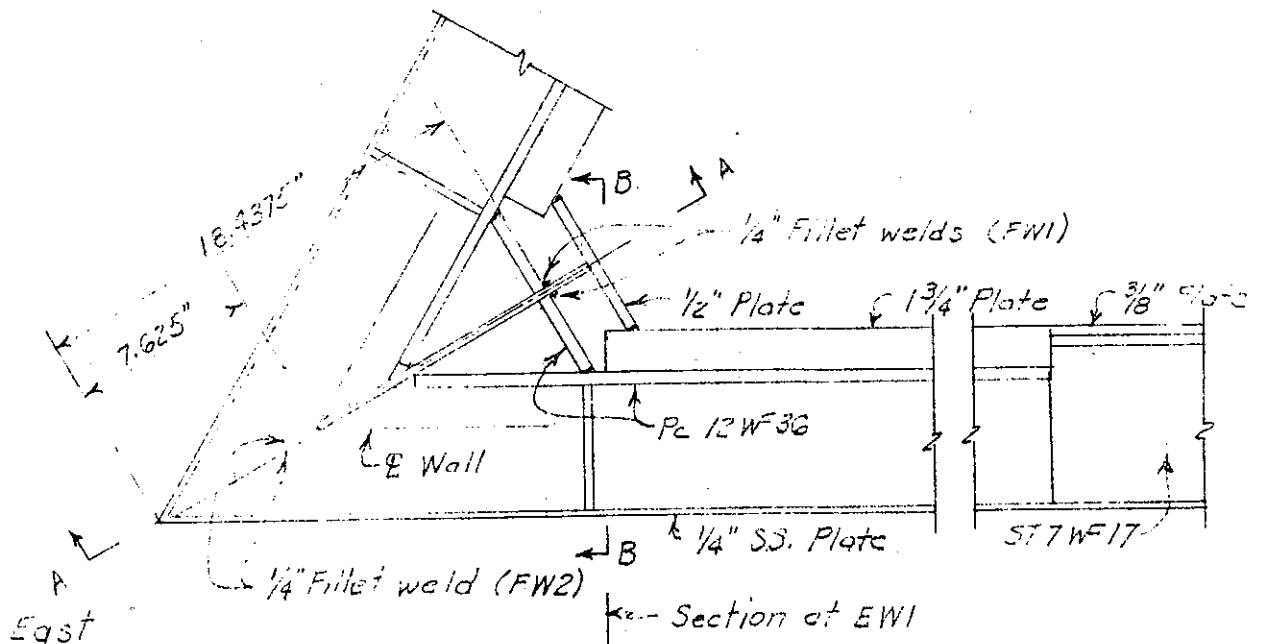
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J.N.E.M. Gateway Arch
Triangular Frames
Station 15

Y91



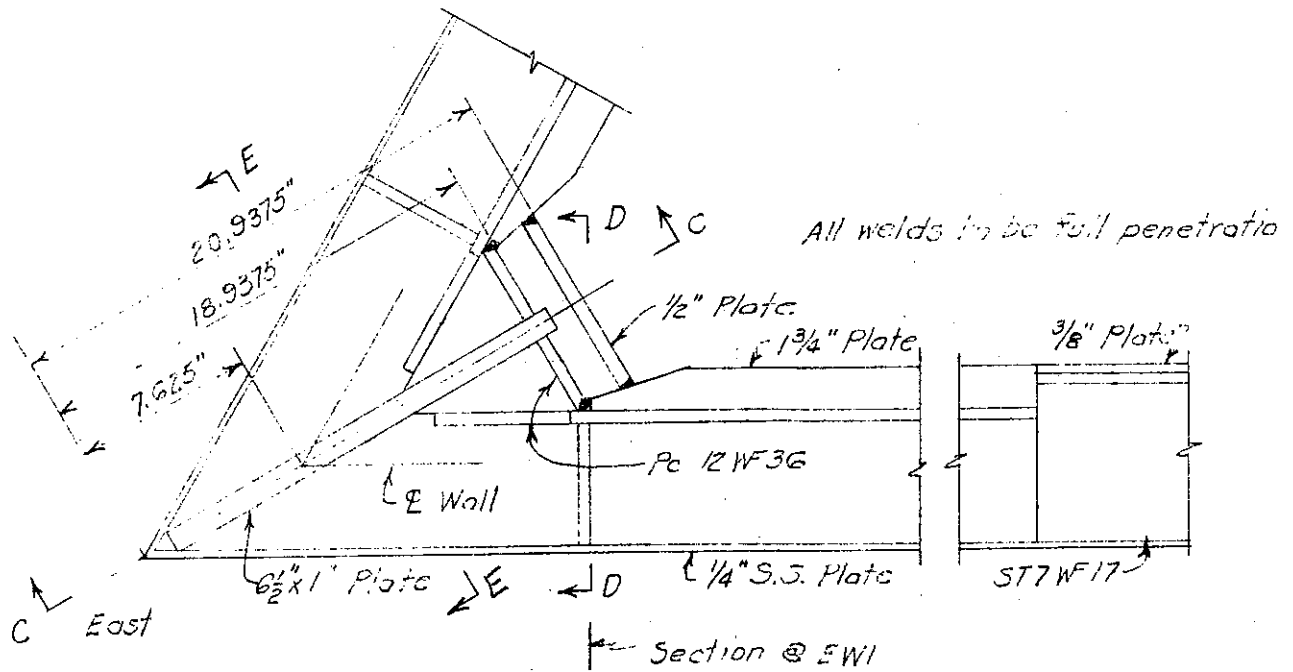
PLAN OF FRAME



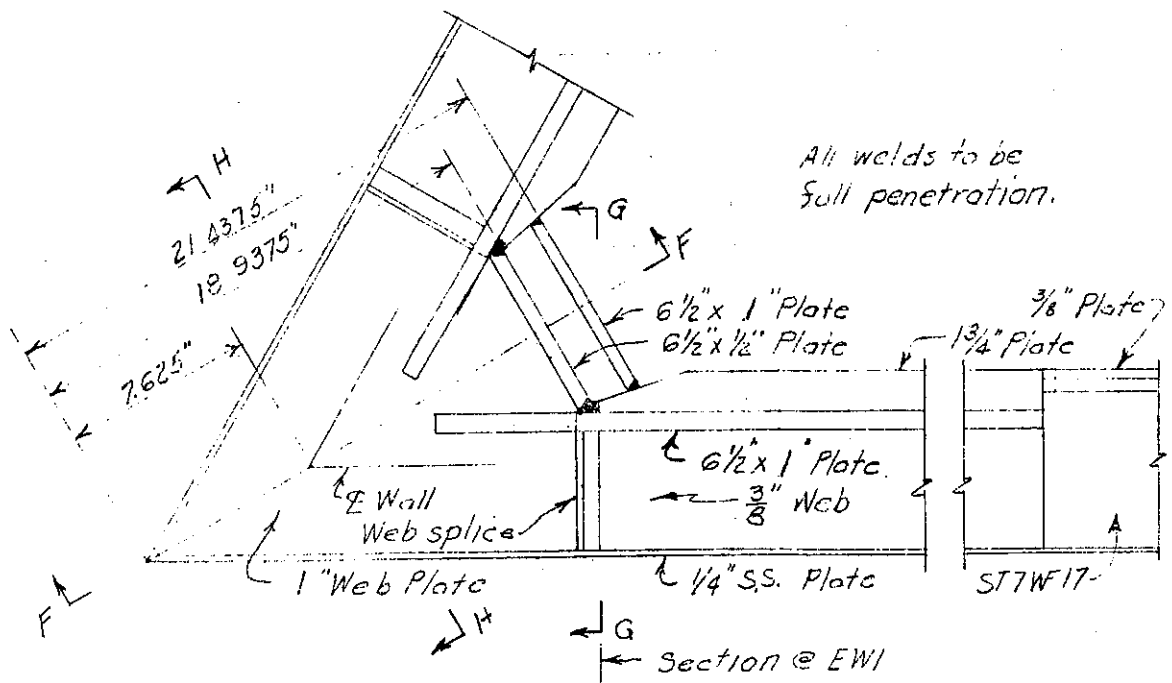
PLAN OF JOINT AS DETAILED

(Pittsburgh-Des Moines Steel Co Shop Dwg.)

Station 15



PLAN OF JOINT DETAILS - MODIFICATION #1



PLAN OF JOINT DETAILS - MODIFICATION #2

Summary of Loading ConditionsCasesA1

Dead Load

Max wind velocity = 25 m.p.h.

Outside air temp = -10°F Inside air temp = 60°F Outside steel temp = -5°F Inside steel temp = 55°F Fabrication temp = 70°F A2

Dead Load

Max wind velocity = 60 m.p.h.

Outside air temp = 114°F Inside air temp = 90°F Outside steel temp, Sides = 110°F , extrados = 145°F Inside steel temp, Sides = 90°F , extrados = 55°F Fabrication Temp = 40°F B1

Dead Load

Max wind velocity = 75 mph

Outside air temp = 20°F Inside air temp = 65°F Outside steel temp = 20°F Inside steel temp = 60°F Fabrication Temp = 70°F B2

Dead Load

Max wind velocity = 75 mph

Outside air temp = 95°F Inside air temp = 75°F Outside steel temp, sides = 94°F , extrados = 125°F Inside steel temp, sides = 77°F , extrados = 50°F Fabrication temp = 40°F

A
 Extreme Wind Conditions
 B
 Extreme Wind Conditions
 A

Summary of Loading Conditions ContinuedCasesC1

↑
Normal Condition
↓

Dead Load

Max wind velocity = 50 m.p.h.

Outside air temp = 20°F

Inside air temp = 65°F

Outside steel temp = 20°F

Inside steel temp = 60°F

Fabrication temp = 70°F

C2

Dead Load

Max wind velocity = 50 mph

Outside air temp = 90°F

Inside air temp = 75°F

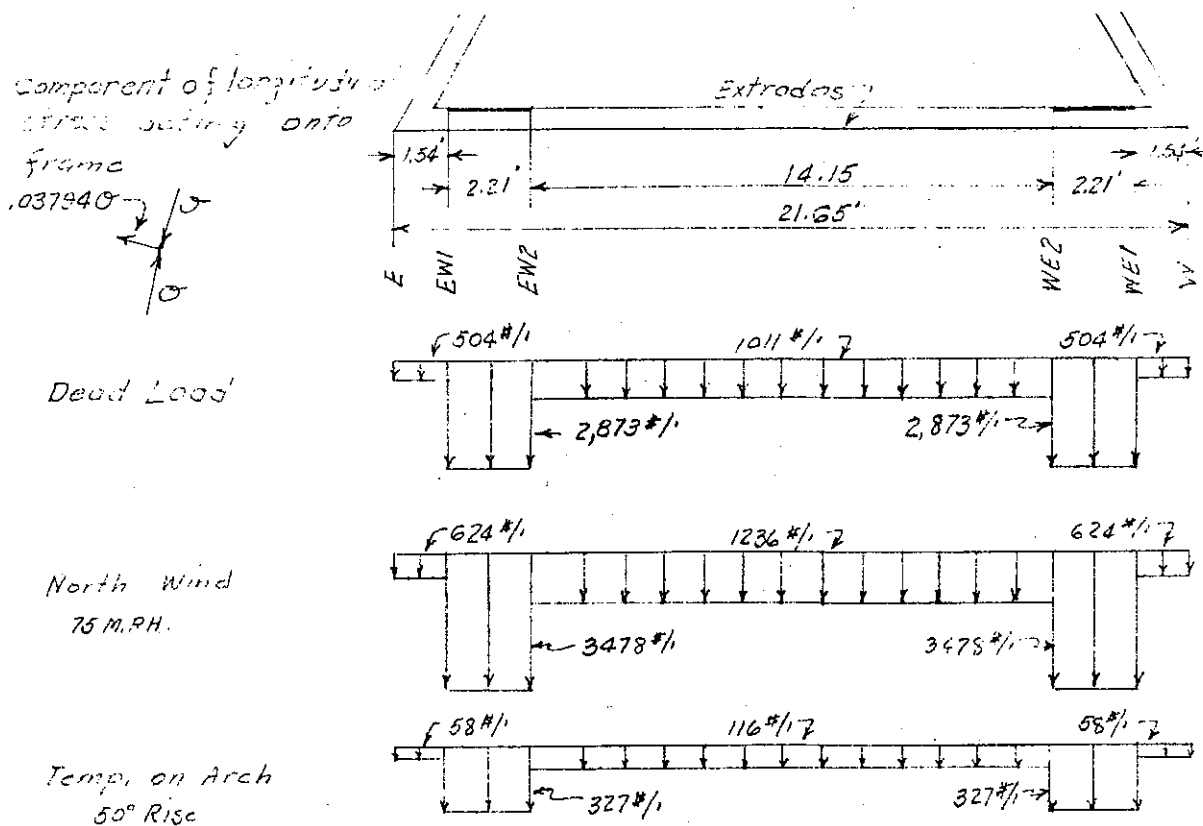
Outside steel temp, sides = 90°F, extrados = 125°F

Inside steel temp, sides = 77°F, extrados = 80°F

Fabrication temp = 40°F

Station 5

Transverse Forces carried by Triangular Frame Extrados North Leg.

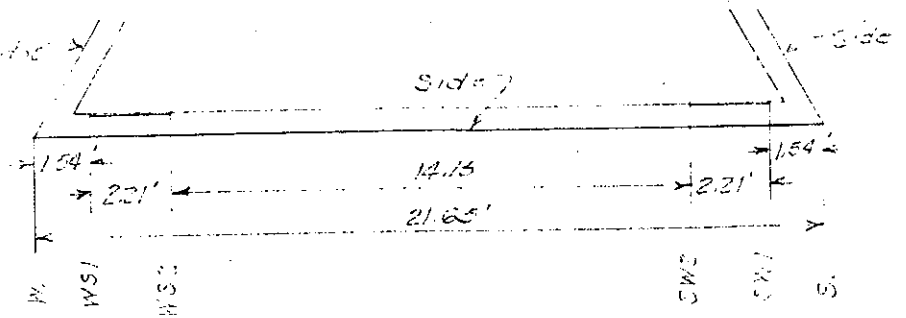


IV f6

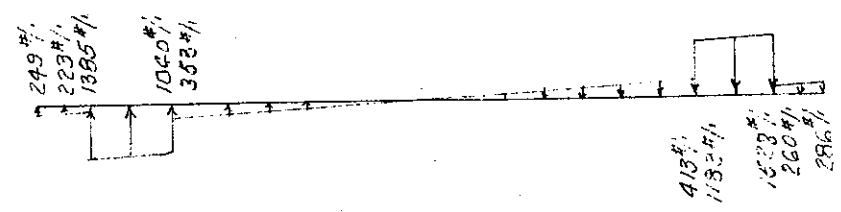
Station 0

Transverse Forces carried by Triangular Frame, Sides North Log.

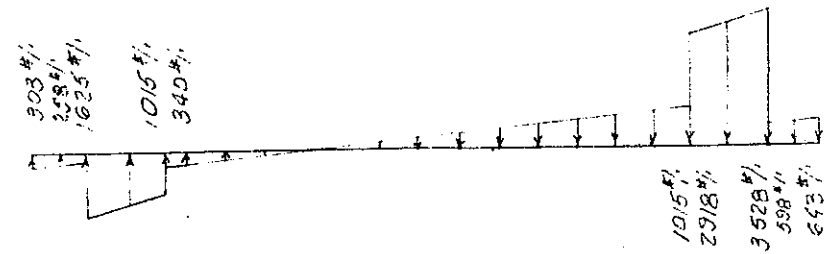
Component of normal stress acting on frame joints.



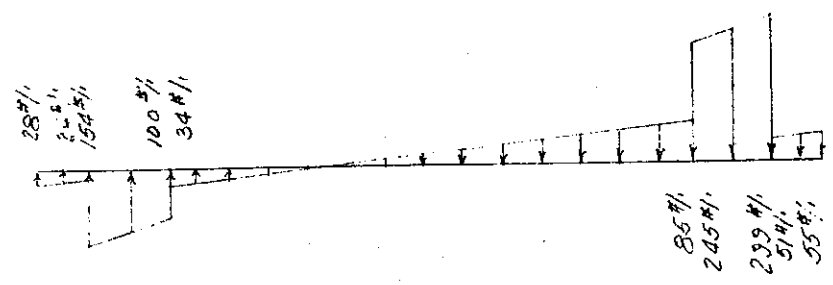
Dead Load

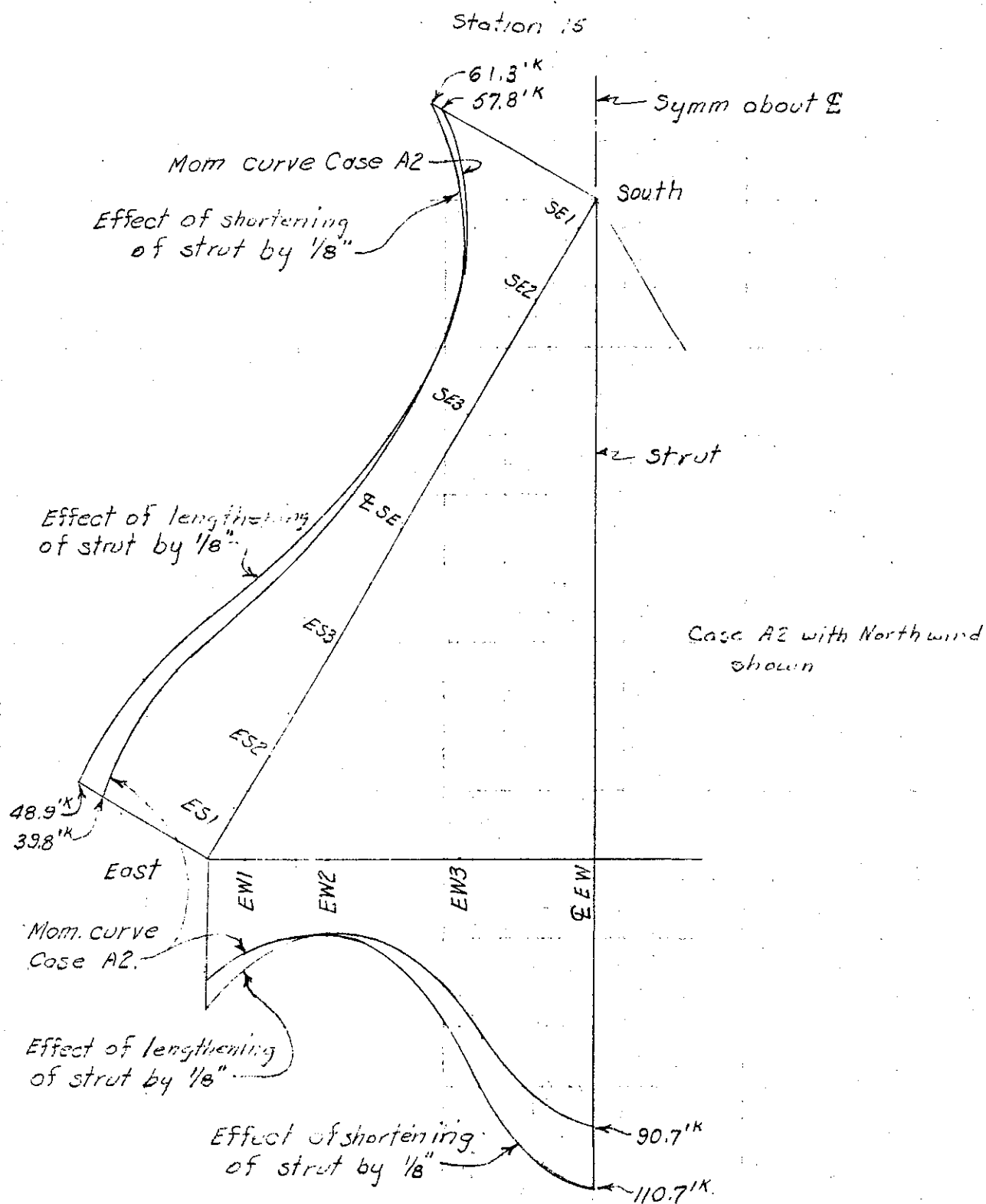


North Wind
75 M.P.H



Temp on Arch
50° Rise



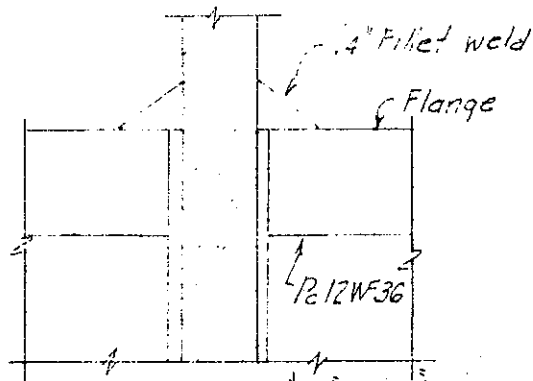


FRAME MOMENT CURVES AT STATION 15
SHOWING EFFECT OF STRUT DEFORMATION

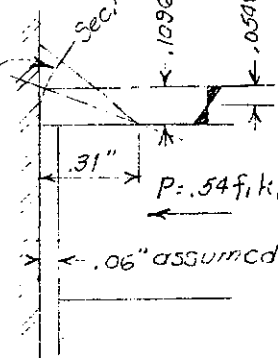
Station 15

Check Section A-A at South Joint Cont.

check weld on flange



Assume depth
of developed
section .31 x .679
= .211"



Assume moment from eccentricity
distributed as shown which is
equiv. to assuming both the plate
and flange as rigid.

K_1 assumed to be .9

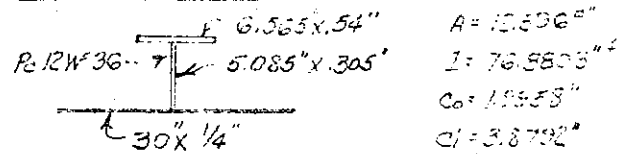
Section X $P/A = .943 (.54 fi \times .9) / .211$

$$M_c/I = (.54 fi \times .9) (.0548 \times 6) / .211^2$$

$$f_{max} = \frac{.54 fi (.9)}{.211} \left\{ .943 + \frac{6 (.0548)}{.211} \right\}$$

$$= 5.76 fi \text{ Say } 6 fi$$

Max stress in weld = 112,000 psi.

Check Section B-B at SEI

$$M = 44348' \# \quad P = 9455' \text{ Comp}$$

$$P/A = 751 \text{ psi c}$$

$$M/S = 44348 \times 12 \times 1.9958 / 76.8803$$

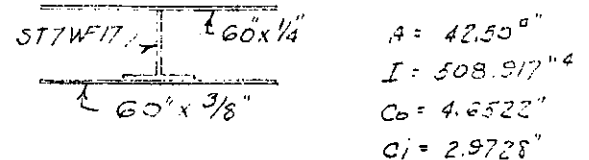
$$= 13815 \text{ psi Comp O.S.}$$

$$44348 \times 12 \times 3.792 / 76.8803$$

$$= 26852 \text{ psi tens I.S. Pl}$$

$$f_t = 26101 \text{ psi I.S. plate}$$

$$f_c = 14566 \text{ psi O.S. plate}$$

Check Section of E SE

$$M = 36813' \# \quad P = 1127' \text{ Comp}$$

$$P/A = 27 \text{ psi comp.}$$

$$M/S = 4038 \text{ psi c O.S., } 2580 \text{ psi t I.S.}$$

$$f_t = 2553 \text{ psi I.S. plate}$$

$$f_c = 4065 \text{ psi O.S. plate}$$

Check Section B-B at ESI

Properties of section same
as at SEI.

$$M = 41537' \# \quad P = 11558 \text{ tens.}$$

$$P/A = 918 \text{ psi tens.}$$

$$M/S = 12940 \text{ psi c. O.S., } 25130 \text{ psi t I.S.}$$

$$f_t = 26068 \text{ psi I.S. plate}$$

$$f_c = 12022 \text{ psi O.S. plate}$$

Station 15

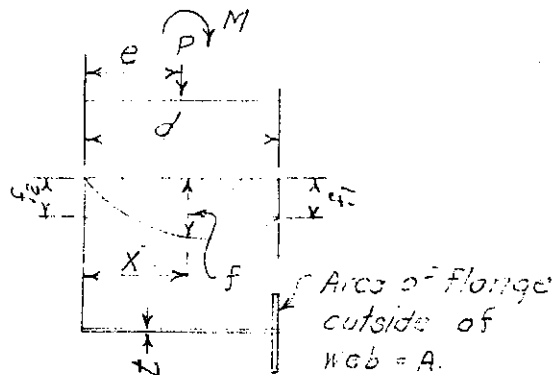
THE FOLLOWING CALCULATIONS
ARE BASED ON CASE A2
LOADING CONDITIONS.

For Moment Diagram at Station 15
see Chart 25.7

Joint is Detailed

Check Section A-A at Joint
(See Sheet 25.1 for Joint Details)

The 1/4" skin plate and 1/2" corner
plates shown on Plan of Joint
are not effective.



Assume a parabolic distribution
of stresses

$$f = f_2(4x/d)(1-x/d) + (x/d)^2 f_1$$

f_1 and f_2 are shown as compression

Avg stress over flange = $k f_1$

$$\sum V = 0, \int f_2 dt/3 + f_1 dt/2 + k f_1 A = P$$

$$f_2 = (3/2dt) [-f_1 (dt/2 + kA) + P]$$

$$f_2 = (3P/2dt) - (3/4 + 3kA/2dt) f_1$$

$$\sum M = 0, \int f_2 dt (3kd/2) + (f_1 dt/2)(2d/3) + k f_1 A d = M + Pe$$

$$(f_1 - f_2)(kd^2t/3) + k f_1 A d = M + Pe$$

$$[f_1 - (3/4 + 3kA/2dt) f_1 + 3P/2dt] d^2t/3 + k f_1 A d = M + Pe$$

$$f_1 [(1 - 3/4 - 3kA/2dt) d^2t/3 + kAd] + Pd/2 = M + Pe = [d^2t/12 - kAd/2 + kAd] + Pd/2$$

$$f_1 = [M + P(e-d/2)] / [(d/2)(dt/6 + kA)]$$

Case A2 $M = -57821$ $P = -7313$ tons

$d = 18.4375$ $t = .305$ $A = 3.3804$ $e = 7.635$
Assume $k = .900$

$$f_1 = \frac{-12(-57821) - 7313(-1.6937)}{9.21875(18.4375 \times .305 + 3.3804 \times .9)} = 18597 \text{ psi. tens.}$$

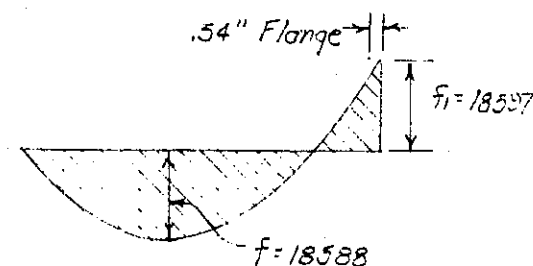
$$f_2 = \frac{3(-7313)}{2(18.4375 \times .305)} - \left(\frac{.75 + 3(3.3804)(.9)}{2(18.4375 \times .305)} \right) (-18597) = 27089 \text{ psi comp.}$$

$$f = \frac{27089(4x)}{18.4375} \left(\frac{1-x}{18.4375} \right) + \frac{x^2(-18597)}{18.4375}$$

when $f = 0$ $x = 15.2731$

f at $x = 7.6365 = 18588 \text{ psi comp.}$

k assumed = .900 Actual = .901 ok.



Final Stress Distribution Sec. A-A

Station 15

Check Section A-A at East Joint

For formulas see Section A-A
of South Joint Sheet I f 8.

$$M = -39828' \# \quad P = -4398 \# \text{ Tension}$$

Assume $K = .900$

$$f_1 = 12836 \text{ psi. tens.}$$

$$f_2 = 18870 \text{ psi. comp.}$$

$$f_{10} \quad x = 15.3021"$$

$$f \text{ at } x = 36.511" = 12998 \text{ psi. comp.}$$

$K \text{ actual} = .900 \text{ ok}$

$$\text{Weld stress} = 78000 \text{ psi.}$$

Check Section B-B at EW1

Properties of section same as
at SE1 Sheet I f 9.

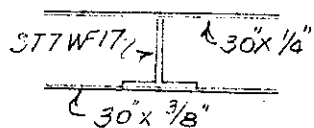
$$M = 32055' \# \quad P = 3156 \text{ Comp.}$$

$$P/A = 251 \text{ psi. comp.}$$

$$M/S = 7986 \text{ psi c. os, } 19409 \text{ psi t. i.s.}$$

$$f_t = 19158 \text{ psi i.s. plate}$$

$$f_c = 10237 \text{ psi o.s. plate}$$

Check Section at EEW

$$A = 23.75 \text{ in}^2$$

$$I = 267.4642 \text{ in}^4$$

$$C_o = 4.7625 \text{ in}$$

$$C_i = 2.8625 \text{ in}$$

$$M = 90,729' \# \quad P = 2682 \# \text{ Tens}$$

$$P/A = 113 \text{ psi. tens.}$$

$$M/S = 19386 \text{ psi c. os, } 11652 \text{ psi t. i.s.}$$

$$f_t = 11765 \text{ psi i.s. plate}$$

$$f_c = 19273 \text{ psi o.s. plate}$$

The stresses shown for Sections
at SE1, ESE, ESI, EW1 and SEW
do not include the stress due to
difference in coefficient of
expansion of materials which
are tabulated in Appendix B

Station 15

Summary of Transverse Frame Stresses - Station 15 (Joint as detailed See Sheet I f. 1)					
Case		Section		Section	
		Section A-A South		Section A-A East	
A1	Base Metal ↑ stress ↓	fc = 18500 I.S.	ft = 20800 Web	fc = 12800 I.S.	ft = 15200 Web
A2		15400 Web	18600 I.S.	13000 Web	12800 I.S.
B1		15100 I.S.	17900 Web	10200 I.S.	11700 Web
B2		19500 Web	20200 I.S.	12600 Web	12800 I.S.
C1		14500 I.S.	15200 Web	9000 I.S.	10700 Web
C2		13600 Web	13200 I.S.	10100 Web	9800 I.S.
		Section B-B SE1		Section B-B EW1	
A1		fc = 34800 I.S.	ft = 17900 O.S.	fc = 27900 I.S.	ft = 13500 O.S.
A2		15000 O.S.	25400 I.S.	10900 O.S.	18100 I.S.
B1		25400 I.S.	14800 O.S.	22500 I.S.	10900 O.S.
B2		14800 O.S.	26100 I.S.	9500 O.S.	17000 I.S.
C1		25700 I.S.	12900 O.S.	20900 I.S.	10000 O.S.
C2		10400 O.S.	17100 I.S.	7800 O.S.	13300 I.S.
		Section B-B ES1		Section @ E EW	
A1		fc = 20700 I.S.	ft = 12200 O.S.	fc = 5800 I.S.	ft = 11300 O.S.
A2		12400 O.S.	25400 I.S.	20100 O.S.	11200 I.S.
B1		17300 I.S.	9400 O.S.	4400 I.S.	8300 O.S.
B2		11700 O.S.	25400 I.S.	18300 O.S.	10300 I.S.
C1		14400 I.S.	8500 O.S.	3700 I.S.	7100 O.S.
C2		9300 O.S.	19600 I.S.	17200 O.S.	9600 I.S.

Stresses are in p.s.i. I.S. = Inside plate O.S. = Outside plate

Note: Stress in welds may be as high as 6 times the base metal stress, depending on restraint of connecting plates. For welds FW1 and FW2 see sheet I f. 1.

The welds on the present design are stressed to failure. These could be improved by making full penetration welds, but base metal is overstressed at several points, even to failure. Therefore no further consideration will be given to the present design.

Station 7 and 19

Summary of Transverse Frame Stresses
(Joint as detailed)

Station 7					
Case	Base Metal Stresses	Section A-A South		Section A-A East	
		fc	ft	fc	ft
A1		19100 I.S.	20100 Web	12000 I.S.	13500 Web
A2		13200 Web	12000 I.S.	11900 Web	11000 I.S.
B1		18400 I.S.	17400 Web	10700 I.S.	12100 Web
B2		13700 Web	12700 I.S.	11700 Web	11300 I.S.
C1		15300 I.S.	15100 Web	9100 I.S.	10500 Web
C2		9000 Web	7800 I.S.	8800 Web	9100 I.S.
Section B-B SEI					
A1		fc 33500 I.S.	ft 20800 O.S.	fc 25400 I.S.	ft 15400 O.S.
A2		12500 O.S.	16800 I.S.	13000 O.S.	20700 I.S.
B1		28000 I.S.	17200 O.S.	28000 I.S.	15000 O.S.
B2		11700 O.S.	15800 I.S.	13200 O.S.	22300 I.S.
C1		25100 I.S.	15300 O.S.	22000 I.S.	12400 O.S.
C2		8400 O.S.	10400 I.S.	9600 O.S.	14900 I.S.
Section B-B ESI					
A1		fc 20500 I.S.	ft 13700 O.S.	fc 6300 I.S.	ft 12400 O.S.
A2		12000 O.S.	19200 I.S.	18500 O.S.	9900 I.S.
B1		17700 I.S.	11000 O.S.	5000 I.S.	9800 O.S.
B2		11200 O.S.	18500 I.S.	17000 O.S.	9200 I.S.
C1		14700 I.S.	9800 O.S.	4300 I.S.	8500 O.S.
C2		9000 O.S.	14500 I.S.	16000 O.S.	6700 I.S.
Station 19					
	Base Metal Stresses	Section A-A South		Section A-A East	
		fc	ft	fc	ft
A1		19500 I.S.	21800 Web	13400 I.S.	16000 Web
A2		21800 Web	21400 I.S.	14800 Web	14500 I.S.
B1		17400 I.S.	17700 Web	10400 I.S.	12000 Web
B2		22500 Web	23200 I.S.	14200 Web	14300 I.S.
C1		14300 I.S.	15300 Web	9300 I.S.	11100 Web
C2		15800 Web	15300 I.S.	11200 Web	10800 I.S.
Section B-B SEI					
A1		fc 36200 I.S.	ft 18800 O.S.	fc 29500 I.S.	ft 14700 O.S.
A2		16900 O.S.	29700 I.S.	11600 O.S.	19800 I.S.
B1		29900 I.S.	15000 O.S.	22000 I.S.	10800 O.S.
B2		17200 O.S.	30900 I.S.	10800 O.S.	18300 I.S.
C1		24200 I.S.	13300 O.S.	21100 I.S.	10300 O.S.
C2		15000 O.S.	20400 I.S.	8000 O.S.	2500 I.S.

Stations 19, 27 and 37

Summary of Transverse Frame Stresses
(Joint as detailed) Cont.

Case		Section B-B ESI		Section at EEW	
A1		fc= 21300 I.S.	ft= 12800 O.S.	fc= 5500 I.S.	ft= 10300 O.S.
A2		13300 O.S.	27500 I.S.	19100 O.S.	11000 I.S.
B1		16400 I.S.	9600 O.S.	3700 I.S.	7000 O.S.
B2		12900 O.S.	27500 I.S.	17500 O.S.	10200 I.S.
C1		14100 I.S.	8700 O.S.	3300 I.S.	6300 O.S.
C2		10200 O.S.	21400 I.S.	16800 O.S.	9700 I.S.

Station 27					
	Base Metal Stresses ↑ Y	Section A-A South		Section A-A East	
A1		fc= 19900 I.S.	ft= 22400 Web	fc= 14000 I.S.	ft= 16600 Web
A2		24300 Web	23800 I.S.	16200 Web	15600 I.S.
B1		14700 I.S.	15600 Web	11400 I.S.	12300 Web
B2		25300 Web	25800 I.S.	15900 Web	15800 I.S.
C1		13100 I.S.	14600 Web	9900 I.S.	11200 Web
C2		17800 Web	17300 I.S.	12300 Web	11800 I.S.
		Section B-B SEI		Section B-B ESI	
A1		fc= 38100 I.S.	ft= 19500 O.S.	fc= 30100 I.S.	ft= 15700 O.S.
A2		19400 O.S.	35300 I.S.	13100 O.S.	23400 I.S.
B1		28100 I.S.	13700 O.S.	21200 I.S.	11000 O.S.
B2		20000 O.S.	37600 I.S.	12800 O.S.	23300 I.S.
C1		26000 I.S.	12900 O.S.	21000 I.S.	10500 O.S.
C2		14000 O.S.	24800 I.S.	9800 O.S.	17100 I.S.
		Section B-B ESI		Section at EEW	
A1		fc= 22700 I.S.	ft= 15400 O.S.	fc= 4400 I.S.	ft= 8300 O.S.
A2		14300 O.S.	28600 I.S.	17300 O.S.	10100 I.S.
B1		19700 I.S.	10700 O.S.	2400 I.S.	4900 O.S.
B2		13900 O.S.	28300 I.S.	15600 O.S.	9200 I.S.
C1		15900 I.S.	9300 O.S.	2300 I.S.	4500 O.S.
C2		11000 O.S.	22300 I.S.	15400 O.S.	9000 I.S.

Station 37					
	Base Metal Stresses ↑ Y	Section A-A South		Section A-A East	
A1		fc= 20600 I.S.	ft= 20200 Web	fc= 14500 I.S.	ft= 14800 Web
A2		21000 Web	23400 I.S.	14300 Web	15600 I.S.
B1		13800 I.S.	12900 Web	13100 I.S.	13500 Web
B2		22200 Web	25700 I.S.	14500 Web	16100 I.S.
C1		12500 I.S.	12800 Web	10800 I.S.	10800 Web
C2		12200 Web	12700 I.S.	10700 Web	10600 I.S.

Station 37 and 44

Summary of Transverse Frame Stresses
(Joint as detailed) Cont.

Case	Section B-B SEI	Section B-B EWI
A'	fc= 45000 I.S. ft= 21100 O.S.	fc= 34600 I.S. ft= 16000 O.S.
AZ	20900 O.S. 43100 I.S.	14900 O.S. 30500 I.S.
B1	33200 I.S. 14300 O.S.	26900 I.S. 12800 O.S.
B2	22100 O.S. 47000 I.S.	15100 O.S. 32200 I.S.
C1	30800 I.S. 13800 O.S.	24900 I.S. 11500 O.S.
C2	14900 O.S. 30100 I.S.	11000 O.S. 22200 I.S.
	Section B-B ESI	Section at EEW
A1	fc= 28300 I.S. ft= 14700 O.S.	fc= 5000 I.S. ft= 9000 O.S.
AZ	14900 O.S. 33800 I.S.	18000 O.S. 10700 I.S.
B1	26700 I.S. 12800 O.S.	4600 I.S. 8200 O.S.
B2	14800 O.S. 35200 I.S.	17100 O.S. 10300 I.S.
C1	21100 I.S. 10700 O.S.	3400 I.S. 6200 O.S.
C2	11300 O.S. 25800 I.S.	15700 O.S. 9400 I.S.

Station 44

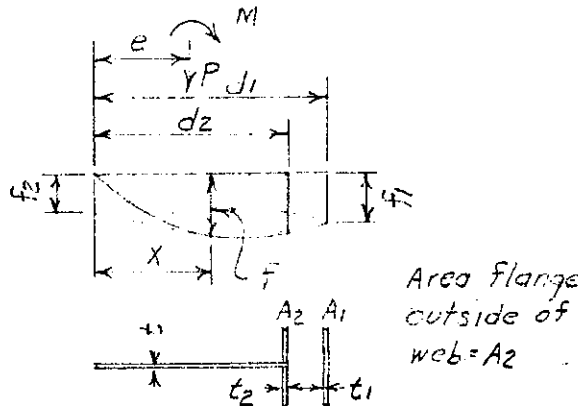
	Section A-A South	Section A-A East
A1	fc= 22700 I.S. ft= 16000 Web	fc= 14500 I.S. ft= 10700 Web
AZ	12000 Web 18100 I.S.	8800 Web 12900 I.S.
B1	15100 I.S. 10500 Web	11500 I.S. 8000 Web
B2	11400 Web 18100 I.S.	8200 Web 12000 I.S.
C1	15000 I.S. 10500 Web	10300 I.S. 7500 Web
C2	8500 Web 12600 I.S.	6600 Web 9600 I.S.
	Section B-B SEI	Section B-B EWI
A1	fc= 37400 I.S. ft= 21100 O.S.	fc= 26500 I.S. ft= 14800 O.S.
AZ	15700 O.S. 26300 I.S.	12400 O.S. 21500 I.S.
B1	27000 I.S. 14000 O.S.	19100 I.S. 10800 O.S.
B2	14700 O.S. 26200 I.S.	11700 O.S. 21100 I.S.
C1	25700 I.S. 13900 O.S.	18400 I.S. 10200 O.S.
C2	11000 O.S. 17900 I.S.	9600 O.S. 16000 I.S.
	Section B-B ESI	Section at EEW
A1	fc= 22400 I.S. ft= 14100 O.S.	fc= 5900 I.S. ft= 10300 O.S.
AZ	12100 O.S. 21700 I.S.	13800 O.S. 8100 I.S.
B1	18200 I.S. 10700 O.S.	4800 I.S. 8300 O.S.
B2	11100 O.S. 21600 I.S.	12600 O.S. 7500 I.S.
C1	15300 I.S. 9800 O.S.	4100 I.S. 7200 O.S.
C2	9100 O.S. 16300 I.S.	11900 O.S. 7100 I.S.

Station 15

Joint with Modifications #1

Check Section C-C at South Joint
(See Sheet Y. & Z for Joint Details)

In all cases the 1/2" corner plate
on Flange of joint is effective.



Assume parabolic distribution
of stresses.

$$f = [4f_2 + f_1]x - 4f_2 x^2 / d_1 / d_1$$

$$\int_0^{d_2} f t dx = (t d_2^2 / d_1) [(4f_2 + f_1) / 2 - 4f_2 d_2 / 3 d_1]$$

$$\int_0^{d_2} f t x dx = (t d_2^3 / d_1) [(4f_2 + f_1) / 3 - f_2 d_2 / d_1]$$

$$f_{d_2} = (d_2 / d_1) [4f_2 (1 - d_2 / d_1) + f_1]$$

$$F_2 = k_2 f_{d_2} A_2 = (k_2 A_2 d_2 / d_1) [4f_2 (1 - d_2 / d_1) + f_1]$$

Moment of F_2 about $x=0$

$$= (k_2 A_2 d_2 / d_1) [4f_2 (1 - d_2 / d_1) + f_1] d_2'$$

$$F_1 = k_1 A_1 f_1 \quad M_0 = k_1 A_1 f_1 d_1'$$

where d_1' and d_2' = $d_1 - t_1/2$
and $d_2 - t_2/2$ respectively.

Equation #1

$$\frac{t d_2^2}{d_1} \left[f_2 \left(2 - \frac{4 d_2}{3 d_1} \right) + .5 f_1 \right] +$$

$$\frac{k_2 A_2 d_2}{d_1} \left[4f_2 \left(1 - \frac{d_2}{d_1} \right) + f_1 \right] + k_1 A_1 f_1 = P$$

Equation #2

$$\frac{t d_2^3}{d_1} \left[f_2 \left(\frac{4}{3} - \frac{d_2}{d_1} \right) + \frac{f_1}{3} \right] + k_2 A_2 \frac{d_2}{d_1} \left[4f_2 \left(1 - \frac{d_2}{d_1} \right) + f_1 \right] \left(d_2 - \frac{t_2}{2} \right) + k_1 A_1 f_1 \left(d_1 - \frac{t_1}{2} \right) = M + P e$$

Note: The moments and stresses
in the stress analysis for
modifications #1 are assumed
to be same as those used in
the As Detailed analysis

$$M = -57821 \text{ lb} \cdot \text{ft} \quad P = -7313 \text{ # tens.}$$

$$d_1 = 20.9375" \quad d_2 = 18.9375"$$

$$d_2 / d_1 = .90448 \quad 1 - d_2 / d_1 = .09552$$

$$t = .305 \quad t_1 = .5 \quad t_2 = .54 \quad e = 7.625"$$

$$A_2 = 3.3804 \text{ in}^2 \quad A_1 = 3.00 \text{ in}^2$$

$$\text{Assume } k_1 = .920 \text{ and } k_2 = .822$$

Solving Eq. 1 & 2

$$5.10867 f_2 + 7.98536 f_1 = -7313$$

$$60.35439 f_2 + 136.77162 f_1 = 74960$$

$$f_1 = 15132 \text{ psi tens}$$

$$f_2 = 21926 \text{ psi comp}$$

$$f = \frac{21926(4x)}{20.9375} \left(1 - \frac{x}{20.9375} \right) + \frac{x(-15132)}{20.9375}$$

$$f = 0 \quad x = 17.3251"$$

$$f \text{ at } x = 8.6626" = 15013 \text{ psi comp}$$

Actual values of k

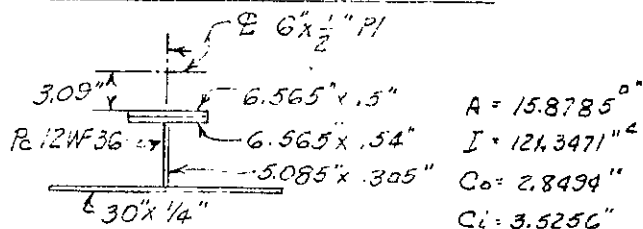
$$k_1 = .920 \text{ and } k_2 = .821 \text{ ok}$$

Stresses on the As same as in

As Detailed

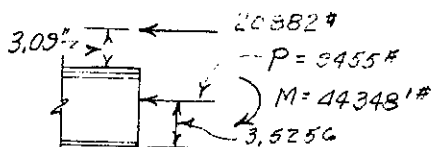
Station 15

Check Section D-D @ SEI



Load applied on 6x1/2" Plate acting normal to section D-D is the horizontal component of stress obtained from section C-C at south joint.

$$= A_1 k_1 f_1 (\sin 30^\circ) = 20882 \#$$



P load at c.g. Section = 30337# Comp.
M at c.g. Section =
44348 (12) - 20882 (3.5256)
= 304029 in-lb

P/A = 1911 psi comp.
M/S = 9252 psi c.o.s., 11448 psi t. i.s.
ft = 9532 psi i.s. plate
fc = 11163 psi o.s. plate

Check Section at E SE

Same as shown on Sheet V f 9
ft = 2553 psi i.s. plate
fc = 4065 psi o.s. plate

Check Section C-C at East Joint

For formulas see Sheet V f 13.

M = -39828 in-lb P = 4398# Tension

Assume $k_1 = .919$ and $k_2 = .818$

Solving Eq. 1 and 2

$$5.10379 f_2 + 7.87013 f_1 = -4398$$

$$60.26716 f_2 + 136.79125 f_1 = -511470$$

$$f_1 = 10499 \text{ psi tens.}$$

$$f_2 = 15328 \text{ psi comp.}$$

$$f = \frac{15328 (12) (1 - \frac{X}{20.9375}) + X (-10499)}{20.9375}$$

$$f = 0 \quad x = 17.3522 \text{ in}$$

$$f \text{ at } x = 8.6761 \text{ in} = 10528 \text{ psi comp.}$$

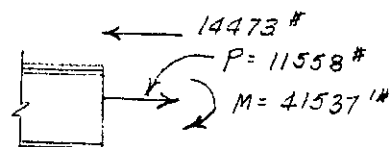
Actual values of K

$$k_1 = .919 \text{ and } k_2 = .818 \quad \text{OK}$$

Check Section D-D at ES1

Properties same as at SEI

Load applied on 6x1/2" plate from section C-C = 14473 #



P load at C.g. Section = 2915# comp.

M at C.g. Section = 402692 in-lb

P/A = 184 psi comp.

M/S = 9456 psi c.o.s., 11700 psi t. i.s.

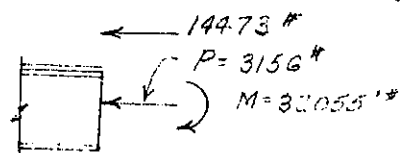
ft = 11516 psi i.s. plate

fc = 9640 psi o.s. plate

Check Section D-D at EW1

Properties same as at SEI

Load applied on 6x1/2" plate from section C-C = 14473 #



P load at C.g. Section = 17529# comp.

M at C.g. Section = 288912 in-lb

P/A = 1110 psi comp.

M/S = 6784 psi c.o.s., 8394 psi t. i.s.

ft = 7284 psi i.s. plate

fc = 7894 psi o.s. plate

Check Section at E EW

Same as shown on V f 10

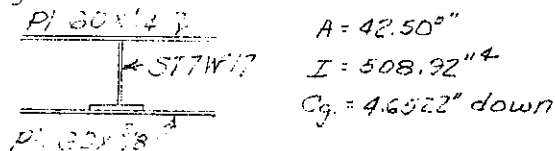
ft = 11765 psi i.s. plate

fc = 10278 psi o.s. plate

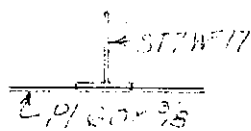
Station 15

Effect of Temperature on Frame

The effective widths of plates used in calculating temperature stresses are the same as used for calculating for frame stresses. The stresses due to temperature are incorporated in the computer program. The method of calculating the stresses are as follows.

Segment of E Sides

$A = 42.50"$
 $I = 508.92"^4$
 $C_g = 4.6922"$ down



$A = 27.50"$
 $I = 53.7137"^4$
 $C_g = 6.8716"$ down
 $r^2 = 1.2260$

Each segment of the sides will produce a local stress in the metal due to difference in coefficient of expansion. Solving for 70° temperature rise and 20° differential

$$217 \cdot 10^3 - \frac{V}{15 \cdot 29 \cdot 10^6} = \frac{V}{27.5 \cdot 29 \cdot 10^6} \left(1 + \frac{6.8716^2}{1.226} \right)$$

$$V = 4135 \#$$

Bottom of \perp $\sigma_c = 277$ psi c.
 Top of \perp $\sigma_c = 6514$ psi t.
 Outside fl. $\sigma_c = 279$ psi c

Due to the difference of strain in extreme fibres the segment will bend to a radius R.

$$\text{Total difference in extreme fibres} = \frac{152000 \cdot (20 \cdot 10^6) / 29 \cdot 10^6}{\dots}$$

$$\text{Radius} = 7.315 \cdot 10^6 / 346.93 = 21,558"$$

For explanation of the preceding calculations see Appendix B for similar calculations.

These segments of various moments of inertia form a beam between the corners of the frame. Since the beams are fixed at the corners a moment is produced which will rotate the curved beam back to tangent at the corners. These end moments will redistribute throughout the frame. These steps were accomplished by a computer program and are not shown in these calculations.

The longitudinal stresses due to temperature are tabulated in Appendix B.

Stresses due to difference in coefficient of expansion of material for Modification #1 are as follows:

Extreme temperature rise

Section	SEI + ESI	EWI	EW
Outer Plate	-451	-678	-751
Inner Plate	-769	-1154	-534

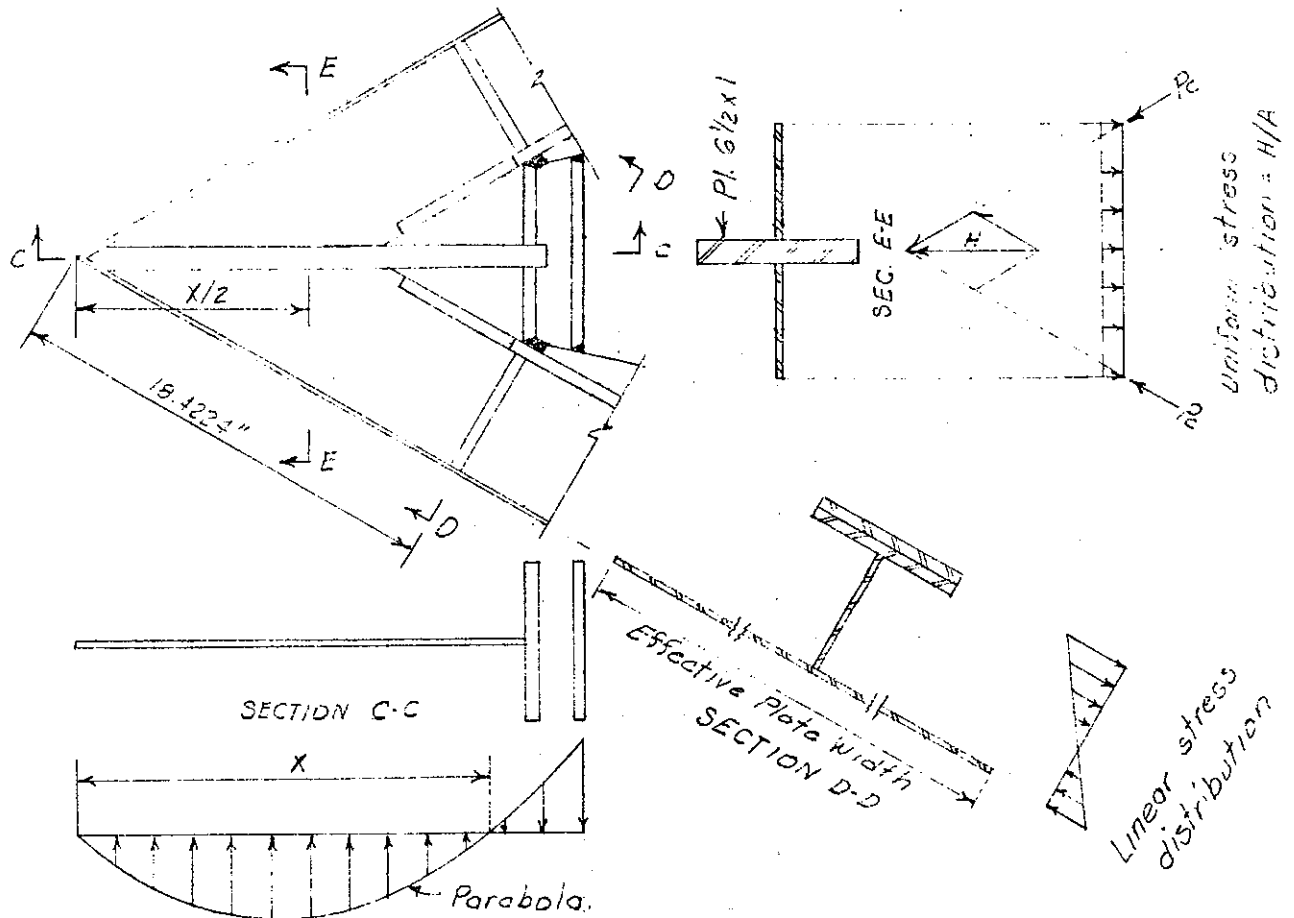
Station 15

Summary of Transverse Frame Stresses - Station 15 (Joint with Modifications #1 See Sheet Yf 2)				
Case		Section		Section
		Section C-C South		Section C-C East
A1	Basic Metal Stresses	fc= 14800 I.S.	Ft= 16600 Web	fc= 10300 I.S. ft= 12300 Web
A2		15000 Web	15100 I.S.	10500 Web 10500 I.S.
B1		13700 I.S.	13900 Web.	8100 I.S. 9400 Web
B2		15100 Web	15200 I.S.	9800 Web 9700 I.S.
C1		11200 I.S.	11900 Web	7200 I.S. 8700 Web
C2		15700 Web	10100 I.S.	7900 Web 7500 I.S.
		Section D-D SEI		Section D-D EWI
A1		fc= 13900 I.S.	Ft= 14700 O.S.	fc= 11600 I.S. Ft= 11600 O.S.
A2		11600 O.S.	8800 I.S.	8600 O.S. 6100 I.S.
B1		11500 I.S.	12000 O.S.	9900 I.S. 9200 O.S.
B2		11900 O.S.	9100 I.S.	7700 O.S. 5900 I.S.
C1		10300 I.S.	10600 O.S.	9000 I.S. 8400 O.S.
C2		8300 O.S.	5900 I.S.	6400 O.S. 4300 I.S.
		Section D-D ESI		Section D EEW
A1		fc= 7100 I.S.	Ft= 9900 O.S.	fc= 5800 I.S. Ft= 11000 O.S.
A2		10100 O.S.	10800 I.S.	20100 O.S. 11200 I.S.
B1		6400 I.S.	7700 O.S.	4400 I.S. 8300 O.S.
B2		9700 O.S.	11100 I.S.	18300 O.S. 10300 I.S.
C1		5000 I.S.	6900 O.S.	8700 I.S. 7100 O.S.
C2		7800 O.S.	8500 I.S.	17200 O.S. 9200 I.S.

Stresses are in p.s.i. I.S. = Inside plate O.S. = Outside plate

Stresses in webs are the same as the base metal stresses.

Station 15

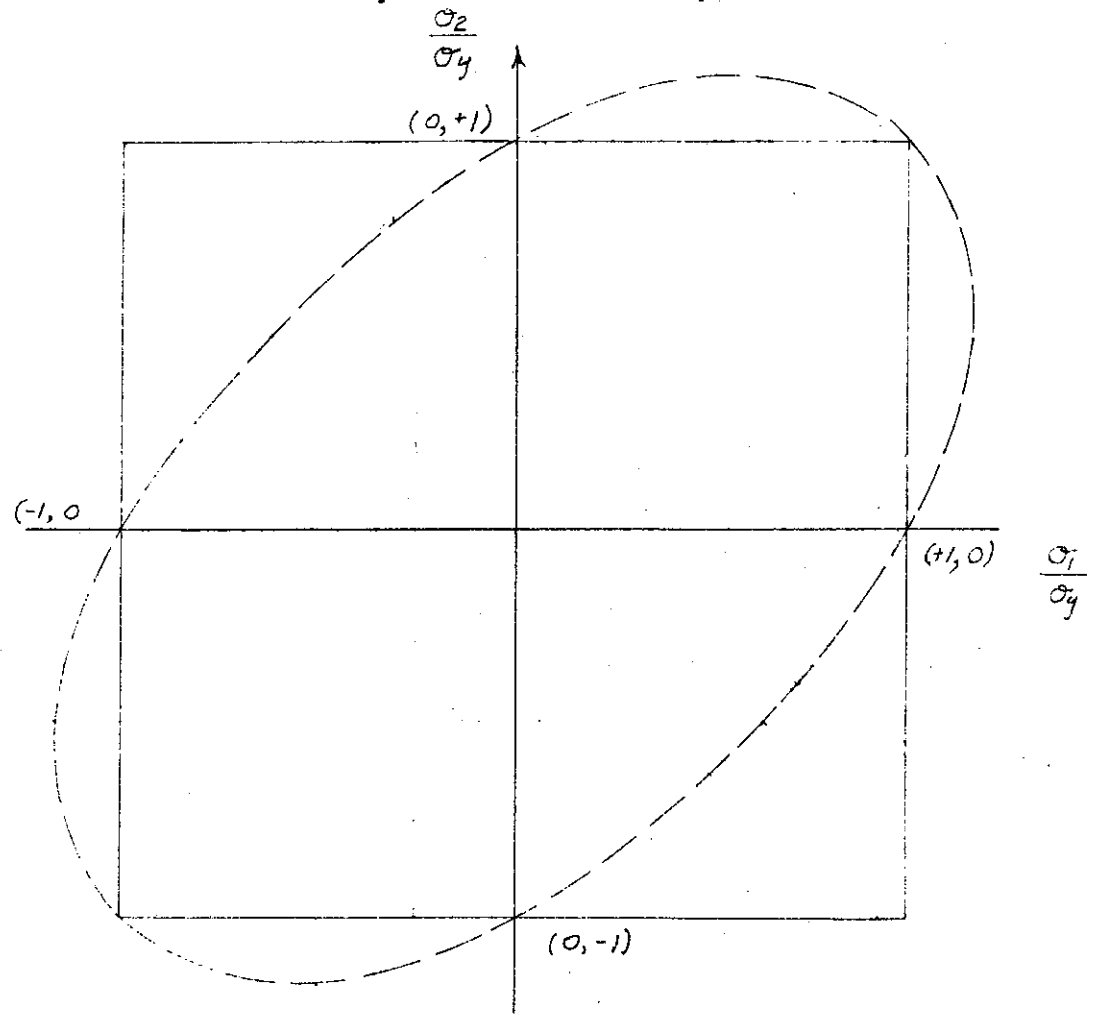
Stresses in Corner Web Plate

Assumed stress distribution

Note: The forces P_e represent the total force in the outside skin plates at Section E-E. To evaluate these, first determine the force in these plates at Section D-D, then reduce the force in linear proportion from D to E assuming the force goes to zero at the apex of the corner.

Shears in section c-c are low and assumed equal to zero. The stresses in Section c-c and E-E are normal to each other and both are considered principal stresses.

Station 15

Failure stresses (Hencky-von Mises theory of failure) σ_1 is the maximum principal stress. σ_2 is the minimum principal stress. σ_y is the yield point stress as determined by the tensile test.

$$\sigma_1^2 + \sigma_2^2 - \sigma_1 \sigma_2 = \sigma_y^2$$

Station 15

Stresses in Corner Web Plate Comp.South Joint

Ave stress in outside plate Section D-D at SEI

$$= 11214 \text{ psi comp. } 1/35(11214 + 8768) / 6.375 = 11214 \text{ psi. comp.}$$

$$P_c \text{ at point E} = 11214 \times 7.5 = 84105 \text{ * Comp.}$$

$$P_c \text{ at point E} = 84105 (36626 / 36603) / 18.4224 = 45666 \text{ * Comp.}$$

$$H = 2fc(18.4224) = 2(45666)(18.4224) = 79596 \text{ * Comp.}$$

$$A = \left\{ \frac{1}{2} [2(36626)(18.4224) - 2(25)(36603)] - \frac{1}{2} \times 300 + 1(6.5) \right\} = 9.0637 \text{ "}$$

+ H = 6.11 psi tension on web & corner plate

From Section D-D $P_c = 15013 \text{ psi. on web.}$

Failure stress from SF20 = 25826 psi.

$$\text{Factor of Safety} = 25826 / 15013 = 1.58$$

East Joint

Ave stress in plate SEI = 10091 = 1/35(10091 + 7471) / 6.375 = 9682 psi Comp.

$$H = 8570 = 1/35(8570 + 8130) / 6.375 = 8284 \text{ psi comp.}$$

$$\text{Total } P_c + P_c' \text{ at point D} = (9682 + 8284) \times 7.5 = 154730 \text{ * Comp.}$$

$$\text{Total } P_c \text{ at point E} = 154730 (36626 / 36603) / 18.4224 = 73267 \text{ * Comp.}$$

$$H = 1/2 \times 73267 (36626) = 63451 \text{ * Comp.}$$

$$A = \left\{ \frac{1}{2} [2(36626)(18.4224) - 2(25)(36603)] - \frac{1}{2} \times 300 + 1(6.5) \right\} = 9.0746$$

+ H = 6992 psi tension on web & corner plate

From Section C-C $P_c = 10528 \text{ psi. on web.}$

Failure stress from SF20 = 22744 psi

$$\text{Factor of Safety} = 22744 / 10528 = 2.16$$

Summary of Stresses and Safety Factors
(Web of Joints with Mod Section #1)

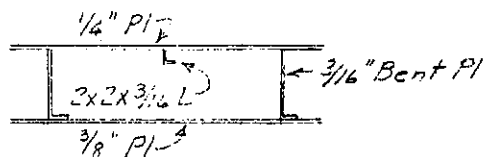
Case	P_c	P_c'	H	H/A	Base Metal Stress	Factor Safety
South Joint						
A1	58600 * t	58600 * t	101500 * t	11100 psi c	16600 psi t	1.66
A2	45700 * c	45700 * c	79100 * c	8700 psi t	15000 psi c	1.58
B1	47100 * t	47100 * t	81500 * t	8900 psi c	13900 psi t	1.65
B2	46300 * c	46300 * c	80200 * c	8800 psi t	15100 psi c	1.57
C1	41900 * t	41900 * t	72500 * t	7900 psi c	11900 psi t	1.91
C2	32900 * c	32900 * c	57000 * c	6200 psi t	10700 psi c	2.22
East Joint						
A1	29500 * t	46500 * t	74900 * t	8200 psi c	12300 psi t	1.94
A2	40400 * c	32900 * c	63500 * c	7000 psi t	10500 psi c	2.16
B1	30900 * t	36700 * t	58500 * t	6400 psi c	9400 psi t	2.39
B2	37900 * c	30300 * c	59100 * c	6500 psi t	9800 psi c	2.33
C1	27800 * t	33600 * t	53200 * t	5800 psi c	8700 psi t	2.61
C2	30700 * c	25000 * c	48600 * c	5300 psi t	7900 psi c	2.86

Station 15

Longitudinal Stresses due to Wind

Maximum loading and stresses
on arch will be at Station 32
Wind load = $1460/32.44 = 45 \#/\text{ft}$

Max Shear on 2' Panel
 $= 2 \times 5.8019 \times 45 = 522 \#$



I of 24" Section = 206.84 in^4
 Checking weld stress $1/4"$ plate
 to $3/16"$ Bent Plate.

$Q = 27.52$
 $VQ/Ib = 522 \cdot 27.52 / 206.84 \times .1875$
 $= 371 \#/\text{in}$

Shear per weld at end
 $371 \times 1.5 \times .1875 = 104 \#$

Shear per weld at 1' from end
 $307.5 \times .1875 = 288 \#$

Stresses due to bending from
 wind load

$M = w \cdot L^2 / 12 = 2 \times 45 \times 11.6038^2 / 12$
 $= 1010.1 \text{ ft} \cdot \text{lb}$

Mc/I 257 psi t on $1/4"$ Pl
 130 psi c on $3/8"$ Pl.

The above stresses are based
 on south wind of 75 mph velocity
 acting on sides. Since the
 stresses are so low they are
 not considered in the frame
 stresses.

Station 15

Summary of Stresses and Safety Factors
(Joints with Modification #1)

Case	Wind	Longitudinal Stresses			Transverse Stresses			Safety Factor Principal Stresses	Safety Factor Buckling
		Bending & Axial	Temp	Total	Bending & Axial	Temp	Total		
1/4" Outside Plate @ SEI									
A1	N	4000 t	14800 t	18800 t	1000 c	14900 t	13900 t	1.95	
	S	2400 t	14800 t	17200 t	200 c	14900 t	14700 t	2.05	
A2	N	7900 t	8300 c	400 c	3400 c	8400 c	11800 c	2.84	4.87
	S	500 c	8300 c	9600 c	1200 t	8400 c	7200 c	3.81	2.80
B1	N	10600 t	9900 t	20500 t	5100 c	9900 t	4800 t	1.77	
	S	3800 c	9900 t	6100 t	2200 t	9900 t	12100 t	3.14	
B2	N	10600 t	6700 c	3900 t	5100 c	6700 c	11800 c	2.33	2.55
	S	3800 c	6700 c	10500 c	2200 t	6700 c	4500 c	3.61	2.73
C1	N	6500 t	9900 t	16400 t	2500 c	9900 t	7400 t	2.31	
	S	100 t	9900 t	10000 t	700 t	9900 t	10600 t	3.19	
C2	N	6500 t	5800 c	700 t	2500 c	5800 c	8300 c	3.80	3.42
	S	100 t	5800 c	5700 c	700 t	5800 c	5100 c	4.08	2.20
1/4" Outside Plate @ ESI									
A1	N	3100 c	15400 t	12300 t	1200 c	10700 t	9500 t	2.95	
	S	2400 c	15400 t	13000 t	700 c	10700 t	10000 t	2.79	
A2	N	4700 c	9200 c	13900 c	2600 c	7600 c	10200 c	2.64	1.74
	S	1000 c	9200 c	10200 c	700 t	7600 c	7500 c	3.30	2.64
B1	N	5900 c	10300 t	4400 t	3500 c	7100 t	3600 t	8.13	
	S	0	10300 t	10300 t	600 t	7100 t	7700 t	3.55	
B2	N	5900 c	7300 c	13200 c	3500 c	6200 c	9700 c	2.78	2.04
	S	0	7300 c	7300 c	600 t	6200 c	5600 c	4.58	3.43
C1	N	4100 c	10300 t	6100 t	2100 c	7100 t	5000 t	5.86	
	S	1500 c	10300 t	8800 t	200 c	7100 t	6900 t	4.12	
C2	N	4100 c	6300 c	10400 c	2100 c	5700 c	7800 c	3.52	2.59
	S	1500 c	6300 c	7800 c	200 c	5700 c	5900 c	4.63	3.45
1/4" Outside Plate @ EWI									
A1	N	3700 c	15500 t	11800 t	500 t	10800 t	11300 t	2.35	
	S	2800 c	15500 t	12700 t	700 t	10800 t	11500 t	2.71	
A2	N	5800 c	16600 c	22400 c	300 c	8100 c	8400 c	1.68	1.40
	S	900 c	16600 c	17500 c	900 t	8100 c	7200 c	2.16	1.79
B1	N	7200 c	12300 t	3100 t	800 c	7200 t	6400 t	5.95	
	S	400 t	12300 t	10700 t	900 t	7200 t	8100 t	3.41	
B2	N	7200 c	14100 c	21300 c	800 c	6700 c	7500 c	1.76	1.47
	S	400 t	14100 c	13700 c	900 t	6700 c	5800 c	2.77	2.29
C1	N	5000 c	10300 t	5300 t	0	7200 t	7200 t	5.10	
	S	1600 c	10300 t	8700 t	800 t	7200 t	8000 t	3.94	
C2	N	5000 c	12100 c	19100 c	0	6300 c	6300 c	1.85	1.45
	S	1500 c	12100 c	13700 c	500 t	6300 c	5500 c	2.50	1.50

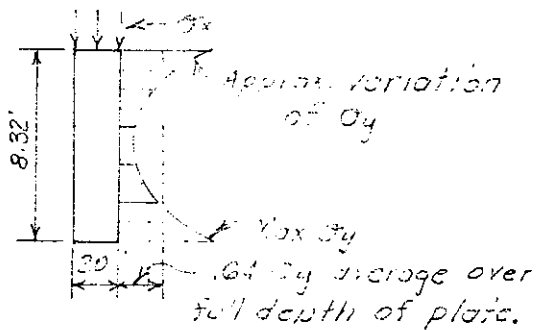
Station 15

Summary of Stresses and Safety Factors Cont (Joints with Modification #1)									
Case		Longitudinal Stresses			Transverse Stresses			Safety Factor Principal Stresses	Safety Factor Backfire
		Bending + Axial	Temp	Total	Bending + Axial	Temp.	Total		
1/4" Outside Plate e EEW									
A1	N	3700 c	11400 t	7700 t	3400 c	13900 t	10500 t	3.56	
	S	2800 c	11400 t	8600 t	2900 c	13900 t	11000 t	3.20	
A2	N	5900 c	12200 c	18000 c	4600 c	15200 c	19800 c	1.74	1.40
	S	900 c	12200 c	13100 c	1800 c	15200 c	17000 c	2.13	1.79
B1	N	7200 c	7400 t	400 t	5300 c	9200 t	3900 t	9.88	
	S	400 t	7400 t	8000 t	1000 c	9200 t	8200 t	4.07	
B2	N	7200 c	10300 c	17500 c	5300 c	13000 c	18300 c	1.84	1.45
	S	400 t	10300 c	9900 c	1000 c	13000 c	14000 c	2.64	2.27
C1	N	5000 c	7600 t	2600 t	4100 c	9200 t	5100 t	7.47	
	S	1600 c	7600 t	6000 t	2200 c	9200 t	7000 t	5.03	
C2	N	5000 c	10300 c	15300 c	4100 c	13100 c	17200 c	2.02	1.64
	S	1600 c	10300 c	11900 c	2200 c	13100 c	15300 c	2.57	1.98
3/8" Inside Plate e EEW									
A1	N	3400 c	7300 c	10700 c	2100 t	7500 c	5400 c	3.56	1.02
	S	2600 c	7300 c	9900 c	1800 t	7500 c	5700 c	3.83	1.63
A2	N	5300 c	7400 t	2100 t	2800 t	8200 t	11200 t	5.56	
	S	1000 c	7400 t	6400 t	1100 t	8200 t	9300 t	4.00	
B1	N	6600 c	4900 c	11500 c	3300 t	5000 c	1700 c	3.06	2.02
	S	200 t	4900 c	4700 c	700 t	5000 c	4300 c	7.31	2.26
B2	N	6600 c	6400 t	200 c	3300 t	7100 t	10400 t	3.74	
	S	200 t	6400 t	6600 t	700 t	7100 t	7800 t	4.53	
C1	N	4200 c	4900 c	9500 c	2500 t	5000 c	2500 c	3.86	2.26
	S	1600 c	4900 c	6500 c	1400 t	5000 c	3600 c	5.35	2.54
C2	N	4200 c	6400 t	1800 t	2500 t	7100 t	9600 t	3.73	
	S	1600 c	6400 t	4800 t	1400 t	7100 t	8500 t	4.47	

Summary of Stresses and Safety Factors for Joints with Modification #2
are identical to those for Joints with modification #1 shown on Sheets
IF 23 and IF 24.

Station 6

Local Buckling of Skin Plate



See pg. 332 - "Theory of Elastic Stability" by Timoshenko.

$$D = 29 \times 10^6 (1.375)^3 / 12$$

$$\sigma_c = \pi^2 D / 12 h^3 = 3.14^2 29 \times 10^6 (1.375)^3 / 12 (8.32 \times 12)^3 (1.375) = 369,769$$

$$a/b = 8.32/2 = 4.16 \quad (a/h)^2 = 17.3056$$

Since σ_y is nearly a direct function of σ_x . Assume $\sigma_y = F \sigma_x$

Using Eq. (2)

$$\sigma_{cr} = \frac{\sigma_c (n^2 - 1 + \frac{1}{2} \pi^2 F^2)}{n^2 + F (1 + \frac{1}{2} \pi^2 F^2)}, \quad n=1$$

$$\sigma_{cr} = \frac{369,769 (1 + 17.3056 F^2)}{1 + F (17.3056)}$$

Solving for Case A:

$$\sigma_x = 20,700 \text{ psi} \quad \sigma_y = 3456 \text{ psi}$$

$$F = 3456 / 20,700 = .167$$

$$\text{For } n=1 \quad \sigma_{cr} = 17,503$$

$$\text{Factor of Safety} = 1.63$$

In order to increase this safety factor a horizontal stiffener will be added to each side of frame.

The variation of σ_y will be determined by the stress distribution. See "Effective Flange Width" - Engineering, Dec. 27, 1946

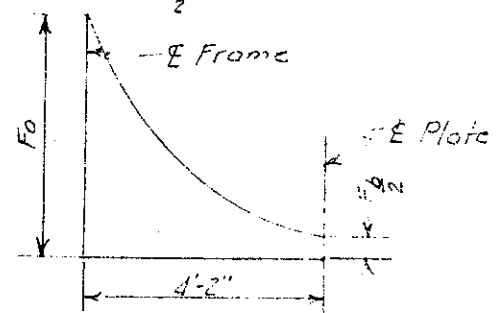
Span length "L" on extrados assumed to be 7' with $a/b = 2.16$ or $7 (10.83) = 7.58'$

Width between Frames $b = 8.32'$

$$F_0 / F_b = \cosh \delta / KL \quad K = .803$$

$$\cosh 8.32 / 363 \times 7.58 = \cosh 6.0238 = 10.312$$

$$F_0 = 100 \quad F_b = 9.7$$



Stress Distribution

Variation of σ_y Ave.

$$1st \ 6" = .90 \sigma_y \quad 1st \ 12" = .80 \sigma_y$$

$$2nd \ 6" = .71 \sigma_y \quad 2nd \ 12" = .61 \sigma_y$$

$$3rd \ 6" = .64 \sigma_y \quad 1st \ 15" = .56 \sigma_y$$

$$\text{Center } 8.32' = .64 \sigma_y$$

$$7.32' = .49 \sigma_y$$

$$6.32' = .36 \sigma_y$$

$$5.32' = .31 \sigma_y$$

The above values are for listed for the extrados side of frame and corners where full stress is not reached. For the intrados

Station 15

Local Buckling of Inside Skin Plate Cont

With horizontal stiffener added
15 inches each side of transverse
diaphragm.

For case A1

$$\sigma_x = 10700 \text{ psi. } \sigma_y = .76 (5400) = 4104$$

$$\sigma_e = 3.14^2 \cdot 29 \cdot 10^6 \cdot (.375)^3 / 10.92 (15)^2 (.375)$$

$$= 16381.6$$

$$a/b = .625 (0.6)^2 = .3906 \quad n=1$$

$$\sigma_{xcr} = \frac{16381.6 (n^2 + .3906)^2}{m^2 + .38355 (.3906)}$$

$$\text{For } m=1 \quad \sigma_{xcr} = 21551$$

Since this value of σ exceeds the
proportional limit this value must
be modified. See table 27 pg. 343
"Buckling strength of metal structures"
by Bleich

$$\sigma_{xcr} = 30119 \text{ psi.}$$

$$\text{Factor of safety} = 2.44$$

For case 3.83 of plate

$$\sigma_x = 10700 \text{ psi. } \sigma_y = .31 (5400) = 1674$$

$$\sigma_e = 755.628$$

$$a/b = 2.91 \quad (0.6)^2 = 8.4681 \quad n=1$$

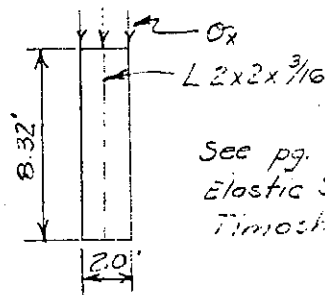
$$\sigma_{xcr} = \frac{755.628 (m^2 + 8.4681)^2}{m^2 + 1.5644 (8.4681)}$$

$$\text{For } m=2 \quad \sigma_{xcr} = 22062$$

$$\text{Factor of safety} = 2.06$$

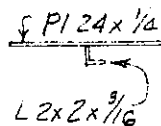
These stiffeners are not used as
safety factor is above 1.40 without
stiffeners. See explanation on
sheet II f 27

Local Buckling of Outside Skin Plate



See pg. 371 "Theory of
Elastic Stability" by
Timoshenko.

check if L 2x2x3/16 will be ample
to force a nodal line in the
plate at the angle.



$$A = 6.71 \text{ in}^2$$

$$C_g = 28.95 \text{ in. from}$$

$$I \text{ of angle at } C_g$$

$$= 1.6528 \text{ in}^4$$

$$a = 100 \text{ in. } b = 24 \text{ in. } \beta = a/b = 4.16$$

$$R = EI / 6 E h^3 / 12 (1 - \nu^2)$$

$$= E \cdot 1.6528 / 24 E (.25)^3 / 10.92 = 48.129$$

$$\beta_0 = A / 10 h = .71 / 24 (.25) = .1153$$

Since $\beta > 2$ Use Eq. 225, pg 375

$$\sigma_{cr} = k E^2 D / b^2 h$$

$$k = [(1 + 17.3056) + 2(28.129)] /$$

$$17.3056 (1 + 23.66)$$

$$= 20.156$$

The value of k is greater than 1
therefore the angle is ample to
force a nodal line in the plate
at the angle. The angle remains
straight while the plate buckles.

$$\sigma_{cr} = 16 (3.14)^2 \cdot 29 \cdot 10^6 \cdot (.25)^3 / 24^2 (.25) 109$$

$$= 45,500 \text{ psi}$$

Since this value of σ_{cr} exceeds
the proportional limit this value
must be modified.

Station 15

Local Buckling of Outside Skin Plate Cor

See pg 360 - "Buckling Strength of Metal Structures" by Bleich.

For $\lambda = .1183$ and $a/b = 4.16$

ϕ Req'd = 29.5 or a Max value of 38 for all cases.

$$I_o \text{ Req'd} = .092 b t^3 \phi = 1.311 < I$$

$$\text{Therefore } \frac{\sigma_{cr}}{\gamma T} = 14.46 E (t/b)^2$$

$$= 45,500 \text{ psi}$$

Assuming the steel with proportional limit of 25,000 psi and yield point of 33,000 psi.

From table pg 313 $\sigma_{cr} = 30080 \text{ psi}$.

Checking Local Stability of stiffener angle see pg. 380

$$c/b = 1 \quad t_c/t = 1$$

$$\text{Web } \frac{\sigma_{cr}}{\gamma T} = \pi^2 E (t/b)^2 K / (12(1-\nu^2))$$

$$= (3.14)^2 \cdot 29 \cdot 10^6 \cdot (.09836)^2 K / 10.92$$

$$= 253580 K$$

Since $K \approx 4$ (Table 32) the

value of $\sigma_{cr} = 33000 \text{ psi}$

Web of stiffener is ok.

$$\text{Flange } \frac{\sigma_{cr}}{\gamma T} = .425 \pi^2 E (t_c/c)^2 / 10.92$$

$$= .425 (3.14)^2 \cdot 29 \cdot 10^6 \cdot (.09836)^2 / 10.92$$

$$= 107770$$

$$\sigma_{cr} = 32430 \text{ psi}$$

Flange of stiffener ok

Since the transverse stresses in the corner plate of the extrados are high the buckling safety factor will be low. In order to improve this condition transverse stiffeners will be added.

Try adding transverse stiffeners at 6" ctrs on Extrados

Case A2 at EWI

$$\sigma_x = 22400 \quad .99 \sigma_y = 7560$$

$$\sigma_e = 3.14^2 \cdot 29 \cdot 10^6 \cdot (.25)^3 / 10.92 (6)^2 (.25)$$

$$= 45504.5$$

$$a/b = .5 \quad (a/b)^2 = .25 \quad m = 1$$

$$\sigma_{cr} = \frac{45504.5 (m^2 + .25)^2}{m^2 + .3375 (.25)} = 66528$$

$$\sigma_{cr} = 31500 \text{ psi} \quad F.o.S. = 1.40$$

This factor of safety cannot be increased without increasing the thickness of the outside skin plate. Therefore the need of the transverse stiffener on the inside skin plate is not needed to maintain the overall factor of safety of 1.40.

In order to maintain a safety factor of 1.40 at EWI the max spacing of transverse stiffeners between frames will be 6". These stiffeners are to extend from corner to 1st longitudinal beam plate stiffener beyond 1st corner plate.

Case A2 at EEW (North wind)

Try 1st stiffener @ 9" from E of frame

$$\sigma_x = 18000 \quad .85 \sigma_y = 16830$$

$$\sigma_e = 20224.2$$

$$a/b = .76 \quad (a/b)^2 = .5625$$

$$\sigma_{cr} = \frac{20224.2 (m^2 + .5625)^2}{m^2 + .9351 (.5625)}$$

$$m = 1 \quad \sigma_{cr} = 32357$$

$$\sigma_{cr} = 27696 \text{ psi modified}$$

$$\text{Factor of safety} = 1.53 \quad \text{ok}$$

Station 15

Try 2nd stiffener @ 24" from E of frame

$$\sigma_x = 18000 \quad .51 \sigma_y = 10098$$

$$\sigma_c = 7280.73$$

$$a/b = 1.25 \quad (a/b)^2 = 1.5625$$

$$\sigma_{cr} = \frac{7280.73 (m^2 + 1.5625)^2}{m^2 + .5610 (1.5625)}$$

$$m = 1 \quad \sigma_{cr} = 25476$$

$$\sigma_{cr} = 25217 \text{ psi. modified}$$

$$\text{Factor of safety} = 1.40$$

Interior 52" of plate.

$$\sigma_x = 18000 \quad .19 \sigma_y = 3762$$

$$\sigma_c = 605.829$$

$$a/b = 4.3333 \quad (a/b)^2 = 18.7777$$

$$\sigma_{cr} = \frac{605.829 (m^2 + 18.7777)^2}{m^2 + .2090 (18.7777)}$$

$$m = 3 \quad \sigma_{cr} = 36168$$

$$\sigma_{cr} = 28624 \text{ psi. modified}$$

$$\text{Factor of safety} = 1.59$$

Case A2 at ESI (NW)

Try using m stiffener.

$$.64 \sigma_y = 6528$$

$$D = 20 \cdot 10^6 (1.25)^3 / 10.92$$

See pg 379 "Theory of Elastic Stability" by Timoshenko

$$\sigma_{cr} = \frac{\pi^2 D (1 + \beta^2)^2}{b^2 h \beta^2} + 2800^3$$

$$\beta = a/b = .24 \quad \beta^2 = 48.123$$

$$\sigma_{y cr} = 6956 \text{ psi}$$

In order to increase the factor of safety against buckling. Two horizontal stiffeners will be used, spaced same as on extrados at E

$$\sigma_x = 13900 \quad .51 \sigma_y = 6202$$

$$\sigma_{cr} = \frac{7280.73 (m^2 + 1.5625)^2}{m^2 + .3742 (1.5625)}$$

$$m = 1 \quad \sigma_{cr} = 30169$$

$$\sigma_{cr} = 27042 \text{ psi. modified}$$

$$\text{Factor of safety} = 1.92$$

Case A2 at SE1

$$\sigma_x = 4000 \quad .51 \sigma_y = 6018$$

$$\sigma_{cr} = \frac{7280.73 (m^2 + 1.5625)^2}{m^2 + .5610 (1.5625)}$$

$$m = 1 \quad \sigma_{cr} = 1550$$

$$\text{Factor of safety} = 4.87$$

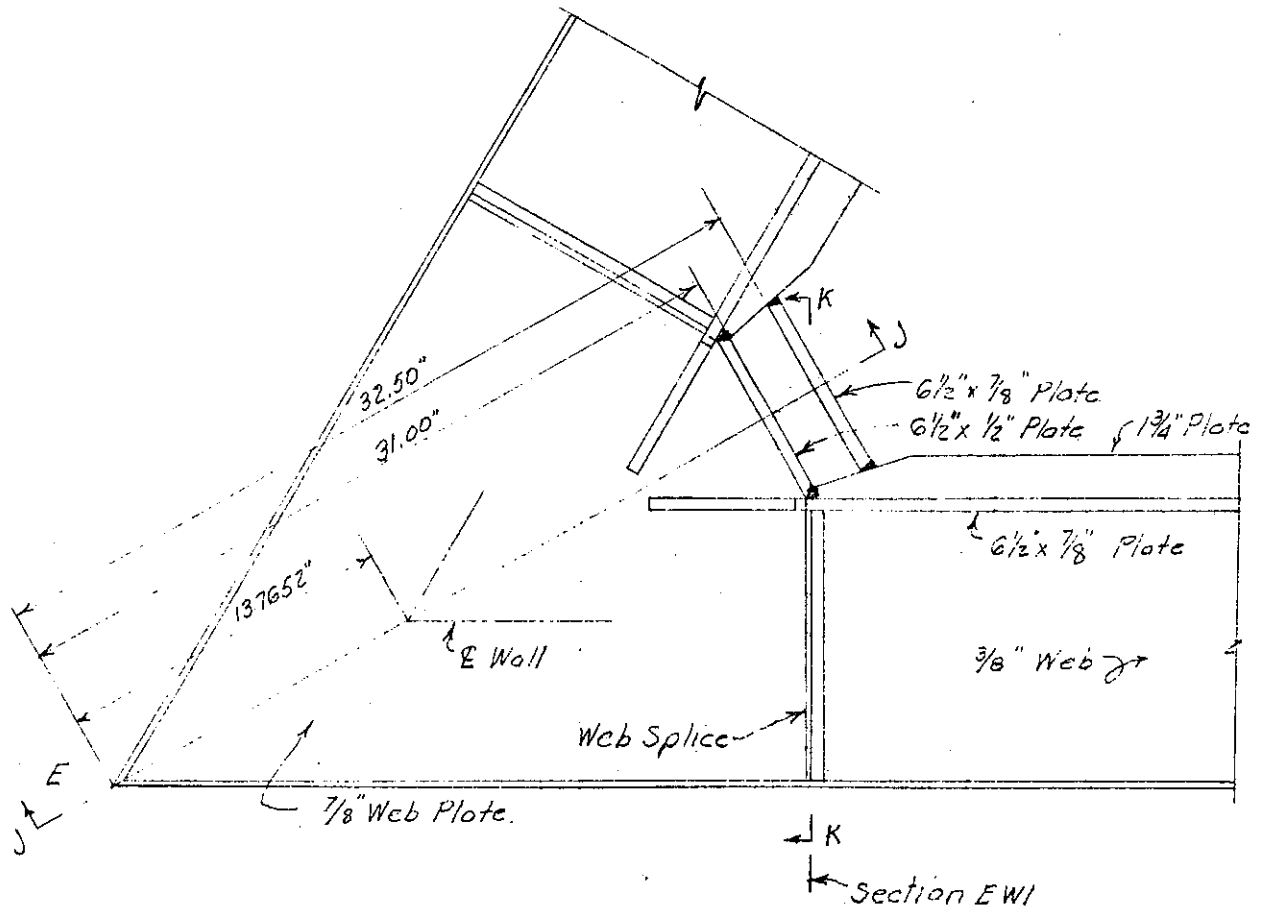
Station 15

Summary of Transverse Frame Stresses - Station 15 (Joint with Modification #2 See Sheet V#2)									
Case		Section F-F South				Section F-F East			
A1	Base metal stress ↑ ↓	fc	7600 I.S.	ft	5000 Web	fc	6200 I.S.	ft	3700 Web
A2			4500 Web		7400 I.S.		3200 Web		5100 I.S.
B1			7100 I.S.		4200 Web		4100 I.S.		2900 Web
B2			4650 Web		7900 I.S.		3000 Web		5000 I.S.
C1			5800 I.S.		3600 Web		3700 I.S.		2600 Web
C2			3300 Web		5200 I.S.		2400 Web		3900 I.S.
		Section G-G SEI				Section G-G EWI			
A1		fc	9500 I.S.	ft	14600 O.S.	fc	8100 I.S.	ft	11700 O.S.
A2			11500 O.S.		6400 I.S.		8500 O.S.		3700 I.S.
B1			8100 I.S.		11900 O.S.		7200 I.S.		9200 O.S.
B2			11400 O.S.		6700 I.S.		7700 O.S.		3600 I.S.
C1			7400 I.S.		10500 O.S.		6400 I.S.		8500 O.S.
C2			8000 O.S.		4200 I.S.		6400 O.S.		2300 I.S.
		Section G-G ESI				Section G-G EW			
A1		fc	4400 I.S.	ft	9800 O.S.	fc	5800 I.S.	ft	11000 O.S.
A2			10200 O.S.		8100 I.S.		20100 O.S.		11200 I.S.
B1			4300 I.S.		7700 O.S.		4400 I.S.		8300 O.S.
B2			9700 O.S.		8400 I.S.		15300 O.S.		10300 I.S.
C1			3100 I.S.		6800 O.S.		3700 I.S.		7100 O.S.
C2			7800 O.S.		6400 I.S.		17200 O.S.		5600 I.S.

Stresses in p.s.i. Stresses in welds are the same as base metal stresses

Summary of Stresses and Safety Factors (Web of Joints with Modification #2)						
Case	P_c	P_c'	H	H/A	Base Metal Stress	Factor Safety
South Joint						
A1	59900 ^{#t}	59900 ^{#t}	95100 ^{#t}	10600 psi c	5000 psi t	2.33
A2	42400 ^{#c}	42400 ^{#c}	73500 ^{#c}	8300 psi t	4500 psi c	2.92
B1	43600 ^{#t}	43600 ^{#t}	75500 ^{#t}	8600 psi c	4200 psi t	2.91
B2	41500 ^{#c}	41500 ^{#c}	72000 ^{#c}	8300 psi t	4600 psi c	2.91
C1	39100 ^{#t}	39100 ^{#t}	67600 ^{#t}	7600 psi c	3600 psi t	3.31
C2	29700 ^{#c}	29700 ^{#c}	51400 ^{#c}	5800 psi t	3300 psi c	4.15
East Joint						
A1	44400 ^{#t}	37600 ^{#t}	71000 ^{#t}	7800 psi c	3700 psi t	3.23
A2	37600 ^{#c}	31700 ^{#c}	60000 ^{#c}	6800 psi t	3200 psi c	3.74
B1	34900 ^{#t}	29000 ^{#t}	55400 ^{#t}	6100 psi c	2900 psi t	4.15
B2	35600 ^{#c}	28300 ^{#c}	55300 ^{#c}	6300 psi t	3000 psi c	4.01
C1	32400 ^{#t}	26000 ^{#t}	50500 ^{#t}	5600 psi c	2600 psi t	4.65
C2	28800 ^{#c}	23700 ^{#c}	45500 ^{#c}	5300 psi t	2400 psi c	5.74

Station 44



PLAN OF JOINT DETAILS - MODIFICATION #2

Station 44

Summary of Transverse Frame Stresses - Station 44
(Joint with Modification #2 See Sheet Vf3c)

Case		Section J-J South		Section J-J East	
A1	Base Metal Stress Y	fc= 11200 I.S.	ft= 6100 Web	fc= 6600 I.S.	ft= 4100 Web
A2		4600 Web	8100 I.S.	3400 Web	5800 I.S.
B1		6800 I.S.	4000 Web	5200 I.S.	3100 Web
B2		4300 Web	8000 I.S.	3100 Web	5500 I.S.
C1		6700 I.S.	4000 Web	4600 I.S.	2900 Web
C2		3300 Web	5600 I.S.	2500 Web	4300 I.S.
		Section K-K SEI		Section K-K EWI	
A1		fc 12300 I.S.	ft= 17800 O.S.	fc= 8800 I.S.	ft= 12700 O.S.
A2		13100 O.S.	8500 I.S.	10900 O.S.	6900 I.S.
B1		9400 I.S.	12000 O.S.	6200 I.S.	9200 O.S.
B2		12300 O.S.	8600 I.S.	10100 O.S.	7000 I.S.
C1		8700 I.S.	11800 O.S.	6100 I.S.	8800 O.S.
C2		9200 O.S.	5700 I.S.	8300 O.S.	5000 I.S.
		Section K-K ESI		Section @ E EWI	
A1		fc= 6700 I.S.	ft= 11900 O.S.	fc= 5900 I.S.	ft= 10300 O.S.
A2		10300 O.S.	7400 I.S.	13800 O.S.	8100 I.S.
B1		5700 I.S.	9000 O.S.	4800 I.S.	8300 O.S.
B2		9500 O.S.	7700 I.S.	12600 O.S.	7500 I.S.
C1		4800 I.S.	8300 O.S.	4100 I.S.	7200 O.S.
C2		7800 O.S.	5600 I.S.	11900 O.S.	7100 I.S.

Stresses in psi. Stresses in welds are the same as base metal stresses.

Summary of Stresses and Safety Factors
(Web of Joints with Modification #2)

Case	P _c	P _c '	H	H/A	Base Metal Stress	Factor Safety
South Joint						
A1	66500 [#] t	66500 [#] t	115200 [#] t	9400psi c	6100psi t	2.43
A2	48500 [#] c	48500 [#] c	84000 [#] c	6900psi c	4600psi c	3.29
B1	44600 [#] t	44600 [#] t	77300 [#] t	6300psi c	4000psi t	3.66
B2	45100 [#] c	45100 [#] c	78000 [#] c	6500psi t	4300psi c	3.49
C1	44200 [#] c	44200 [#] t	76500 [#] t	6200psi c	4000psi t	3.69
C2	34200 [#] c	34200 [#] c	59200 [#] c	4900psi t	3300psi c	4.60
East Joint						
A1	47900 [#] t	45000 [#] t	80400 [#] t	6500psi c	4100psi t	3.56
A2	40800 [#] c	38500 [#] c	68700 [#] c	5600psi t	3400psi c	4.20
B1	34200 [#] t	33700 [#] t	58800 [#] t	4800psi c	3100psi t	4.30
B2	37400 [#] c	35300 [#] c	63000 [#] c	5200psi t	3100psi c	4.65
C1	32900 [#] t	31200 [#] t	55500 [#] t	4500psi c	2900psi t	5.13
C2	31100 [#] c	29000 [#] c	52000 [#] c	4200psi t	2500psi c	5.56

Station 44

Summary of Stresses and Safety Factors (Joints with Modification #2)									
Case	Wind	Longitudinal Stresses			Transverse Stresses			Safety Factor Principal Stresses	Safety Factor Buckling
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total		
1/4" Outside Plate @ SE1									
A1	N	4900 t	14800 t	19700 t	800 c	18100 t	17300 t	1.77	
	S	5900 t	14800 t	20700 t	400 c	18100 t	17700 t	1.70	
	E	5500 t	14800 t	20300 t	400 c	18100 t	17700 t	1.72	
	W	5300 t	14800 t	20100 t	300 c	18100 t	17800 t	1.75	
A2	N	2400 t	8100 c	5700 c	2700 c	10400 c	13100 c	2.90	2.42
	S	8400 t	8100 c	300 t	700 c	10400 c	11100 c	2.93	2.95
	E	5900 t	8100 c	2200 c	600 c	10400 c	11000 c	3.27	3.16
	W	5000 t	8100 c	3100 c	200 c	10400 c	10600 c	3.49	3.07
B1	N	700 t	9900 t	10600 t	4000 c	12100 t	8100 t	3.43	
	S	10100 t	9900 t	20000 t	900 c	12100 t	11200 t	1.90	
	E	6200 t	9900 t	16100 t	700 c	12100 t	11400 t	2.30	
	W	4700 t	9900 t	14600 t	100 c	12100 t	12000 t	2.44	
B2	N	700 t	6600 c	5900 c	4000 c	8300 c	12300 c	3.09	2.78
	S	10100 t	6600 c	3500 t	900 c	8300 c	9200 c	2.90	3.79
	E	6200 t	6600 c	400 c	700 c	8300 c	9000 c	3.74	3.64
	W	4700 t	6600 c	1900 c	100 c	8300 c	8400 c	4.32	4.13
C1	N	3300 t	9900 t	13200 t	2000 c	12100 t	12100 t	2.76	
	S	7500 t	9900 t	17400 t	600 c	12100 t	11500 t	2.15	
	E	5800 t	9900 t	15700 t	500 c	12100 t	11600 t	2.34	
	W	5100 t	9900 t	15000 t	300 c	12100 t	11800 t	2.41	
C2	N	3300 t	5700 c	2400 c	2000 c	7200 c	9200 c	3.99	3.17
	S	7500 t	5700 c	1800 t	600 c	7200 c	7800 c	3.73	4.47
	E	5800 t	5700 c	100 t	500 c	7200 c	7700 c	4.25	4.53
	W	5100 t	5700 c	600 c	300 c	7200 c	7500 c	4.57	4.64
1/4" Outside Plate @ ES1									
A1	N	4100 c	15400 t	11300 t	600 c	12200 t	11600 t	2.88	
	S	4600 c	15400 t	10800 t	500 c	12200 t	11700 t	2.92	
	E	3100 c	15400 t	12300 t	300 c	12200 t	11900 t	2.72	
	W	5600 c	15400 t	9800 t	700 c	12200 t	11500 t	3.06	
A2	N	3000 c	9000 c	12000 c	1400 c	8500 c	9900 c	2.97	2.67
	S	5800 c	9000 c	14800 c	1000 c	8500 c	9500 c	2.54	2.11
	S	3100 t	9000 c	5900 c	400 t	8500 c	8100 c	4.54	4.15
	W	11600 c	9000 c	20600 c	1800 c	8500 c	10300 c	1.84	1.52
B1	N	2200 c	10300 t	8100 t	1900 c	8100 t	6200 t	4.49	
	S	6600 c	10300 t	3700 t	1200 c	8100 t	6900 t	5.51	
	E	7300 t	10300 t	17600 t	900 t	8100 t	9000 t	2.16	
	W	15700 c	10300 t	5400 c	2600 c	8100 t	5500 t	3.49	
B2	N	2200 c	1200 c	9400 c	1900 c	6900 c	6800 c	3.62	3.21
	S	7000 c		13500 c	1200 c	6900 c	8100 c	3.76	

Station 44

Summary of Stresses and Safety Factors Cont. (Units with Modification #2)									
Case	Wind	Longitudinal Stresses			Transverse Stresses			Safety Factor Principal Stresses	Safety Factor Buckling
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total		
1/4" Outside Plate @ ESI									
B2	E	1300 t	7200 c	100 t	900 t	6900 c	6000 c	5.45	5.81
	W	15700 c	7200 c	22900 c	2600 c	6900 c	9500 c	1.65	1.37
C1	N	3400 c	11300 t	6900 t	1100 c	8100 t	7000 t	4.74	
	S	5300 c	10300 t	5000 t	800 c	8100 t	7300 t	5.10	
	E	300 t	10300 t	11100 t	200 t	8100 t	8300 t	9.30	
	W	9400 c	10300 t	900 t	1400 c	8100 t	6700 t	5.23	
C2	N	3400 c	6200 c	9600 c	1100 c	6300 c	7400 c	3.78	3.25
	S	5300 c	6200 c	11500 c	800 c	6300 c	7100 c	3.23	2.72
	E	800 t	6200 c	5400 c	200 t	6300 c	6100 c	5.70	5.74
	W	9400 c	6200 c	15600 c	1400 c	6300 c	7700 c	2.44	2.01
1/4" Outside Plate @ EWI									
A1	N	4700 c	15100 t	10700 t	200 c	12500 t	12300 t	2.84	
	S	5300 c	15400 t	10100 t	100 t	12500 t	12600 t	2.85	
	E	3800 c	15400 t	11600 t	200 t	12500 t	12700 t	2.70	
	W	6200 c	15400 t	9200 t	0	12500 t	12500 t	2.84	
A2	N	3400 c	16400 c	19800 c	1400 c	9500 c	10900 c	1.92	1.59
	S	6700 c	16400 c	23100 c	100 c	9500 c	9600 c	1.64	1.36
	E	2000 t	16400 c	14400 c	600 t	9500 c	8900 c	2.62	2.17
	W	11800 c	16400 c	28200 c	600 c	9500 c	10100 c	1.33	1.11
B1	N	2400 c	10300 t	7900 t	2300 c	8400 t	6100 t	4.60	
	S	7700 c	10300 t	2600 t	200 c	8400 t	8200 t	4.54	
	E	5900 t	10300 t	16200 t	800 t	8400 t	9200 t	2.34	
	W	15700 c	10300 t	8400 c	1000 c	8400 t	7400 t	2.96	5.87
B2	N	2400 c	13900 c	16300 c	2300 c	7900 c	10200 c	2.31	1.92
	S	7700 c	13900 c	21600 c	200 c	7900 c	8100 c	1.74	1.45
	E	5900 t	13900 c	8000 c	800 t	7900 c	7100 c	4.34	3.88
	W	15700 c	13900 c	29600 c	1000 c	7900 c	3900 c	1.25	1.06
C1	N	3900 c	10300 t	6400 t	1000 c	8400 t	7400 t	4.74	
	S	6200 c	10300 t	4100 t	0	8400 t	8400 t	4.53	
	E	200 c	10300 t	10100 t	400 t	8400 t	8800 t	3.46	
	W	9700 c	10300 t	600 t	400 c	8400 t	8000 t	4.27	
C2	N	3300 c	13000 c	17800 c	1000 c	7400 c	8400 c	2.13	1.77
	S	6200 c	13000 c	20100 c	0	7400 c	7400 c	1.87	1.56
	E	200 c	13000 c	14100 c	400 t	7400 c	7000 c	2.70	2.52
	W	9700 c	13000 c	23600 c	400 c	7400 c	7800 c	1.58	1.23
1/4" Outside Plate @ E.W.									
A1	N	4700 c	11400 t	6700 t	800 c	11100 t	10300 t	3.64	
	S	5300 c	11400 t	6100 t	1100 c	11100 t	10000 t	3.78	
	E	3800 c	11400 t	5100 t	1100 c	11100 t	10600 t	3.71	

Station 44

Summary of Stresses and Safety Factors Cont. (Joints with Modification #2)									
Case	Wind	Longitudinal Stresses			Transverse Stresses			Safety Factor	Safety
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total	Principal Stresses	Factor Buckling
1/4" Outside Plots @ E.W									
A1	W	50 00 c	11400 t	6400 t	11 00 c	111 00 t	100 00 t	3.76	
A2	N	34 00 c	12200 c	15600 c	200 t	120 00 c	118 00 c	2.34	1.74
	S	67 00 c	122 00 c	189 00 c	18 00 c	120 00 c	138 00 c	1.94	1.38
	E	50 00 c	122 00 c	172 00 c	15 00 c	120 00 c	135 00 c	2.10	1.50
	W	50 00 c	122 00 c	172 00 c	15 00 c	120 00 c	135 00 c	2.10	1.50
B1	N	24 00 c	76 00 t	52 00 t	9 00 t	74 00 t	83 00 t	4.54	
	S	77 00 c	76 00 t	100 c	22 00 c	74 00 t	52 00 t	6.28	
	E	49 00 c	76 00 t	27 00 t	18 00 c	74 00 t	56 00 t	6.30	
	W	49 00 c	76 00 t	27 00 t	18 00 c	74 00 t	56 00 t	6.30	
B2	N	24 00 c	103 00 c	127 00 c	9 00 t	103 00 c	94 00 c	2.89	2.05
	S	77 00 c	103 00 c	180 00 c	22 00 c	103 00 c	125 00 c	2.06	1.46
	E	49 00 c	103 00 c	152 00 c	12 00 c	103 00 c	121 00 c	2.37	1.63
	W	49 00 c	103 00 c	152 00 c	18 00 c	103 00 c	121 00 c	2.37	1.63
C1	N	39 00 c	74 00 t	37 00 t	200 c	74 00 t	72 00 t	5.29	
	S	62 00 c	74 00 t	14 00 t	15 00 c	74 00 t	57 00 t	6.18	
	E	50 00 c	74 00 t	26 00 t	13 00 c	74 00 t	51 00 t	6.22	
	W	50 00 c	74 00 t	26 00 t	13 00 c	74 00 t	51 00 t	6.22	
C2	N	39 00 c	103 00 c	142 00 c	200 c	104 00 c	106 00 c	2.58	1.83
	S	62 00 c	103 00 c	165 00 c	15 00 c	104 00 c	119 00 c	2.23	1.58
	E	50 00 c	103 00 c	153 00 c	13 00 c	104 00 c	117 00 c	2.38	1.63
	W	50 00 c	103 00 c	153 00 c	13 00 c	104 00 c	117 00 c	2.38	1.63
3/8" Inside Plots @ E.W									
A1	N	44 00 c	72 00 c	116 00 c	500 t	64 00 c	59 00 c	3.28	1.50
	S	49 00 c	72 00 c	121 00 c	800 t	64 00 c	56 00 c	3.14	1.37
	E	46 00 c	72 00 c	118 00 c	700 t	64 00 c	57 00 c	3.22	1.37
	W	46 00 c	72 00 c	118 00 c	700 t	64 00 c	57 00 c	3.22	1.37
A2	N	32 00 c	72 00 t	40 00 t	100 c	69 00 t	68 00 c	5.57	
	S	62 00 c	72 00 t	100 00 t	13 00 t	69 00 t	52 00 t	4.25	
	E	46 00 c	72 00 t	26 00 t	10 00 t	69 00 t	79 00 t	4.73	
	W	46 00 c	72 00 t	26 00 t	10 00 t	69 00 t	79 00 t	4.73	
B1	N	23 00 c	48 00 c	71 00 c	500 c	43 00 c	48 00 c	5.25	1.75
	S	71 00 c	48 00 c	119 00 c	16 00 t	43 00 c	27 00 c	3.05	1.78
	E	45 00 c	48 00 c	93 00 c	11 00 t	43 00 c	32 00 c	4.03	2.03
	W	45 00 c	48 00 c	93 00 c	11 00 t	43 00 c	32 00 c	4.03	2.03
B2	N	23 00 c	63 00 t	40 00 t	500 c	60 00 t	55 00 t	6.70	
	S	71 00 c	63 00 t	800 c	12 00 t	60 00 t	76 00 t	4.10	
	E	45 00 c	63 00 t	15 00 t	11 00 t	60 00 t	71 00 t	5.16	
	W	45 00 c	63 00 t	15 00 t	11 00 t	60 00 t	71 00 t	5.16	

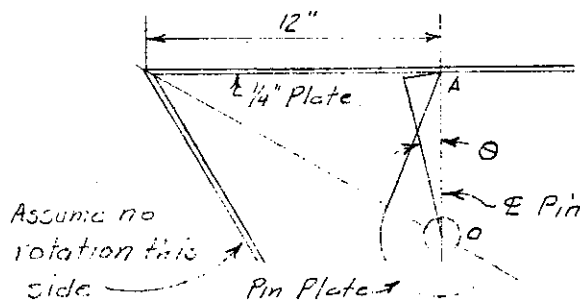
Station 44

Summary of Stresses and Safety Factors Cont (Joint with Modification #2)									
Case	Weld	Longitudinal Stresses			Transverse Stresses			Safety Factor	Safety
		Bending ± Axial	Temp	Total	Bending ± Axial	Temp	Total	Principal Stresses	Factor Buckling
3/8" Inside Plate @ EEW									
C1	N	3600 c	4800 c	8400 c	200 t	4300 c	4100 c	4.53	1.91
	S	5700 c	4800 c	10500 c	1100 t	4300 c	3200 c	3.54	1.87
	E	4600 c	4800 c	9400 c	900 t	4300 c	3400 c	4.00	1.97
	W	4600 c	4800 c	9400 c	900 t	4300 c	3400 c	4.00	1.97
C2	N	3600 c	6300 t	2700 t	200 t	6000 t	6200 t	6.12	
	S	5700 c	6300 t	600 t	1100 t	6000 t	7100 t	4.83	
	E	4600 c	6300 t	1700 t	900 t	6000 t	6900 t	5.29	
	W	4600 c	6300 t	1700 t	900 t	6000 t	6900 t	5.29	

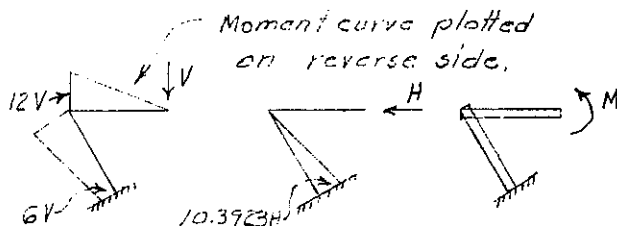
Note: No horizontal stiffeners used on $\frac{3}{8}$ " Inside Plate.
Stiffeners added 12" and 27" each side of frame to 1st longitudinal bent plate stiffener inside of $1\frac{3}{4}$ " corner plates. Also the $\frac{1}{4}$ " Outside plate will have horizontal stiffeners at 6" max spacing from corner to the 1st longitudinal bent plate stiffener inside of $1\frac{3}{4}$ " corner plate.

Station 15

Stresses in Outside Corner Plate: Hinged



$\angle \theta = .01$ Radians (Assumed)
 Distance $AO = 6.6782"$
 $\Delta = 2(AO) \sin(\theta/2) = .066782"$
 $\Delta H = .066781"$
 $\Delta V = .000334"$



Load plate with unit loading at V, H and M and solve.

$$\Delta V = \frac{1584.00V - 495.83H - 180.00M}{EI}$$

$$\Delta H = \frac{-498.53V + 435.00H + 62.35M}{EI}$$

$$\Delta \theta = \frac{-180.00V + 62.35H + 24.00M}{EI}$$

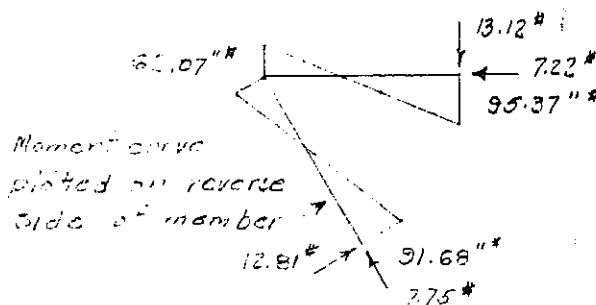
$$EI = 37760$$

Solving the equations

$$V = 13.12 \text{ \#}$$

$$H = 7.22 \text{ \#}$$

$$M = 95.37 \text{ \#}$$



Max stress

$$\frac{7.22}{.25} + \frac{95.37 \times 6}{.25^2} = 9184 \text{ psi}$$

Stresses proportional to rotation.

Case	Max. Rotation	Total Stress*	Safety Factor*
A1	$\theta = .009879$ $\theta' = .009879$	17800 psi	1.85
A2	$\theta = .018097$ $\theta' = .006749$	22600 psi	1.46
B1	$\theta = .008273$ $\theta' = .008273$	14900 psi	2.21
B2	$\theta = .012113$ $\theta' = .006435$	22300 psi	1.47
C1	$\theta = .007167$ $\theta' = .007167$	12900 psi	2.55
C2	$\theta = .015536$ $\theta' = .004596$	19300 psi	1.80

* These values of safety factors and stresses are based on the assumption that the 1/4" plate is fixed at point A. This is not true above or below the pin plate. If the fixity were 75% the minimum safety factor would be 1.94.

Station 15

Station 15 with hinged joints will be calculated without the strut between the Intrados corner and the Extrados side. The outside $\frac{1}{4}$ " skin plate will be stainless steel as used in the as detailed structure. The two horizontal stiffeners used in Modification #1 and #2 will be used and extended to corners in lieu of 6" cts at corners.

Summary of Stresses and Safety Factors. (Hinged Joints)									
Case	Wind	Longitudinal Stresses			Transverse Stresses			Safety Factor Principal Stresses	Safety Factor Buckling
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total		
1/4" Outside Plate @ SE1									
A1	N	4000 t	14800 t	18800 t	1200 t	2100 t	3300 t	1.89	
	S	2100 t	14800 t	17200 t	600 t	2100 t	2700 t	2.05	
A2	N	7200 t	8300 c	400 c	2800 t	1900 c	900 t	-	3.04
	S	1300 c	8300 c	9600 c	600 c	1900 c	2500 c	3.82	
B1	N	10600 t	9900 t	20500 t	3300 t	1400 t	5300 t	1.79	
	S	3800 c	9900 t	6100 t	1400 c	1400 t	0	6.40	
B2	N	10600 t	6700 c	3900 t	3900 t	1500 c	2400 t	9.68	2.75
	S	2800 c	6700 c	10500 c	1400 c	1500 c	2900 c	3.51	
C1	N	6500 t	3900 t	16400 t	2200 t	1400 t	3600 t	2.21	
	S	1000 t	3900 t	10000 t	200 c	1400 t	1200 t	3.48	
C2	N	6500 t	5300 c	700 t	2200 t	1300 c	900 t	-	
	S	1000 t	5300 c	5700 c	200 c	1300 c	1500 c	6.44	
1/4" Outside Plate @ E1									
A1	N	3100 c	15400 t	12300 t	700 t	2100 t	2800 t	2.95	
	S	2400 c	15400 t	13000 t	400 t	2100 t	2500 t	2.76	
A2	N	1700 c	9200 c	13900 c	1700 t	1900 c	200 c	2.39	2.11
	S	1000 c	9200 c	10200 c	0	1900 c	1900 c	3.51	
B1	N	5900 c	10300 t	4400 t	2300 t	1400 t	3700 t	8.05	
	S	0	10300 t	10300 t	300 c	1400 t	1100 t	3.36	
B2	N	5900 c	7300 c	13200 c	2300 t	1500 c	800 t	2.42	2.25
	S	0	7300 c	7300 c	300 c	1500 c	1800 c	5.00	
C1	N	4100 c	10300 t	6100 t	1300 t	1400 t	2700 t	6.23	
	S	1500 c	10300 t	8800 t	200 t	1400 t	1600 t	4.06	
C2	N	4100 c	6300 c	10400 c	1300 t	1400 c	100 c	3.18	2.90
	S	1500 c	6300 c	7800 c	200 t	1400 c	1200 c	4.53	
1/4" Outside Plate @ W1									
A1	N	3700 t	15500 t	11800 t	2800 t	2000 t	4800 t	3.21	
	S	2800 c	15500 t	12700 t	2200 t	2000 t	4200 t	2.94	
A2	N	5800 c	16600 c	22400 c	4300 t	2300 c	1500 t	1.42	1.55
	S	900 c	16600 c	17500 c	1100 t	2300 c	1700 c	1.77	
B1	N	7200 c	10300 t	3100 t	5400 t	1300 t	6700 t	5.28	1.71
	S	200 t	10300 t	10700 t	300 t	1300 t	1600 t	2.00	

Station 15

Summary of Stresses and Safety Factors Cont.
(Hinged Joints)

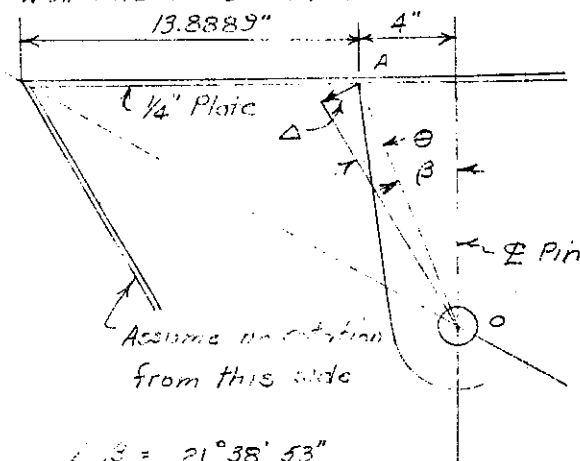
Case	Weld	Longitudinal Stresses			Transverse Stresses			Safety Factor Principal Stresses	Safety Factor Buckling
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total		
1/4" Outside Plate EWI									
B2	N	7200 c	14100 c	21300 c	5400 t	2300 c	3100 t	1.43	1.42
	S	400 t	14100 c	13700 c	300 t	2300 c	2000 c	2.58	2.16
C1	N	5000 c	10300 t	5300 t	3700 t	1300 t	5000 t	6.39	
	S	1600 c	10300 t	8700 t	1600 t	1300 t	2900 t	4.30	
C2	N	5000 c	14100 c	13100 c	3700 t	2300 c	1400 t	1.66	1.58
	S	1600 c	14100 c	15700 c	1600 t	2300 c	700 c	2.14	1.72
1/4" Outside Plate @ EEW									
A1	N	3700 c	11400 t	7700 t	5300 t	200 c	5100 t	4.86	
	S	2800 c	11400 t	8600 t	4300 t	200 c	4100 t	4.42	
A2	N	5800 c	12200 c	16000 c	8000 t	200 t	8200 t	1.42	1.68
	S	900 c	12200 c	13100 c	2400 t	200 t	2600 t	2.26	2.51
B1	N	7200 c	7600 t	400 t	9800 t	100 c	9700 t	3.47	
	S	400 t	7600 t	8000 t	1000 t	100 c	900 t	4.34	
B2	N	7200 c	10300 c	17500 c	9800 t	100 t	9900 t	1.37	1.73
	S	400 t	10300 c	9900 c	1000 t	100 t	1100 t	3.14	3.06
C1	N	5000 c	7600 t	2600 t	7000 t	100 c	6900 t	5.46	
	S	1600 c	7600 t	6000 t	3100 t	100 c	3000 t	6.35	
C2	N	5000 c	10300 c	15300 c	7000 t	100 t	7100 t	1.66	1.98
	S	1600 c	10300 c	11900 c	3100 t	100 t	3200 t	2.35	2.54
3/8" Inside Plate @ EEW									
A1	N	3400 c	7300 c	10700 c	3500 c	500 t	3000 c	3.45	1.97
	S	2600 c	7300 c	9900 c	2500 c	500 t	2300 c	3.67	2.23
A2	N	5300 c	7400 t	2100 t	5200 c	700 c	5900 c	4.59	1.79
	S	1300 c	7400 t	6400 t	1500 c	700 c	2200 c	4.26	5.08
B1	N	6600 c	4900 c	11500 c	6300 c	300 t	6000 c	3.31	1.49
	S	200 t	4900 c	4700 c	700 c	300 t	400 c	7.31	5.15
B2	N	6600 c	6400 t	200 c	6300 c	500 c	6800 c	4.92	1.64
	S	200 t	6400 t	6600 t	700 c	500 c	1200 c	4.53	9.32
C1	N	4600 c	4900 c	9500 c	4500 c	300 t	4200 c	4.50	1.52
	S	1600 c	4900 c	6500 c	2000 c	300 t	1700 c	5.65	3.80
C2	N	4600 c	6400 t	1800 t	4500 c	500 c	5000 c	5.41	2.23
	S	1600 c	6400 t	4800 t	2000 c	500 c	2500 c	5.13	4.57

Station 44

The wall thickness as detailed is .6334' from Station 0 thru 37 then varies to 1.2157' at Sta. 45. In order to accommodate the hinged joint and transfer of stress the wall thickness needs to be increased. The new wall thickness is .6354' from Station 0 thru 27 then varies to 1.2157' at Station 45.

Stress in Outside Corner Plate, Sta. 44

Wall thickness = 1.1670'



$$\angle \beta = 21^\circ 38' 53''$$

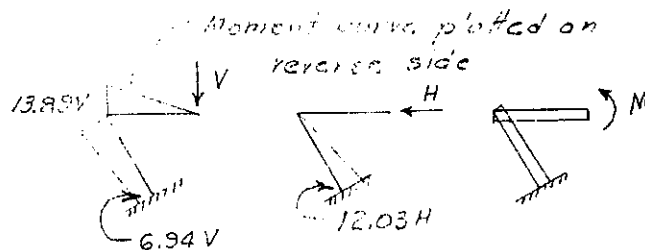
$$\angle \theta = .01 \text{ Radian (Assumed)}$$

$$\text{Distance } AO = 10.842944''$$

$$\Delta = 2 (10.842944) \sin(\theta/2) = .10842914''$$

$$\Delta V = .04050330''$$

$$\Delta H = .10058013''$$



$$\Delta V = \frac{2455.92 V}{EI} - \frac{773.42 H}{EI} - \frac{241.13 M}{EI}$$

$$\Delta H = \frac{-773.42 V}{EI} + \frac{669.80 H}{EI} + \frac{83.53 M}{EI}$$

$$\Delta \theta = \frac{-241.13 V}{EI} + \frac{83.53 H}{EI} + \frac{27.78 M}{EI}$$

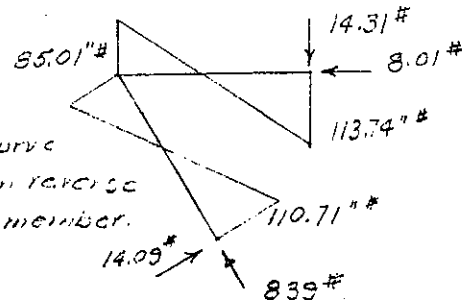
$$EI = 37760$$

Solving the equations

$$V = 14.31 \#$$

$$H = 8.01 \#$$

$$M = 113.74 \# \cdot \text{in}$$



Moment curve plotted on reverse side of member.

Moment Diagram for .01 Radian Rotation Outside Plate.

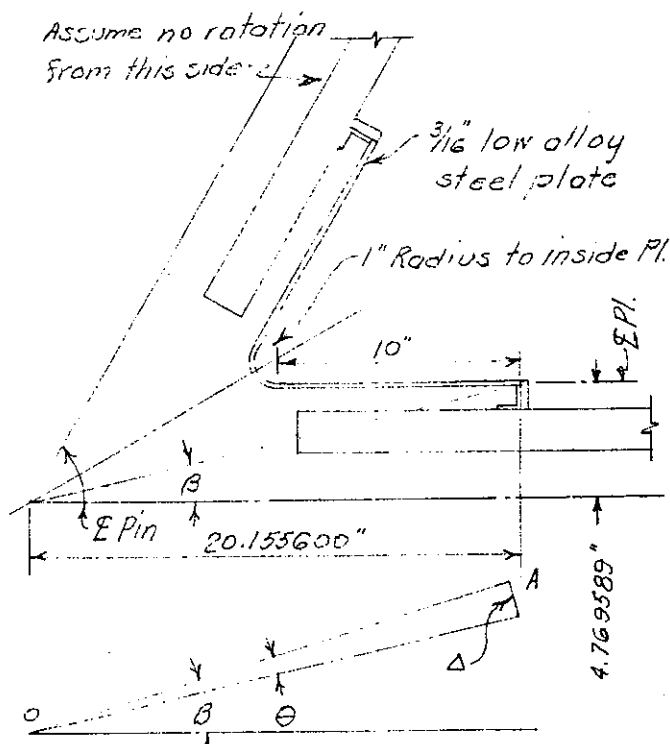
Max Stress

$$\frac{8.01}{.125} + \frac{113.74 \times 6}{.125} = 10901 \text{ p.s.i.}$$

Stresses are proportional to the rotation

Case	Max Rotation	Stress	Safety Factor
A1	$\theta = .006660$ $\theta' = .006660$	14400 p.s.i.	2.29
A2	$\theta = .003965$ $\theta' = .002684$	12700 p.s.i.	2.59
B1	$\theta = .004922$ $\theta' = .004922$	10600 p.s.i.	3.11
B2	$\theta = .009058$ $\theta' = .002261$	12300 p.s.i.	2.68
C1	$\theta = .004600$ $\theta' = .004600$	10000 p.s.i.	3.30
C2	$\theta = .007806$ $\theta' = .002726$	10500 p.s.i.	3.14

Station 44

Stresses in Inside Corner Plate - Hinged

$$\angle \beta = 13^\circ 18' 49''$$

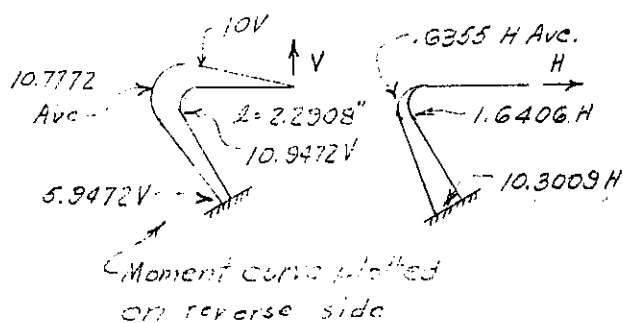
$$\text{Distance } OA = 20.7122$$

$$\angle \theta = .01 \text{ Radian (Assumed)}$$

$$\Delta = 2 (20.7122) \sin(\theta/2) = .207122''$$

$$\Delta H = .046687''$$

$$\Delta V = .201791''$$



Loading plate with unit loading of V, H and H and only.

$$\Delta V = \frac{1333.79V - 480.28H + 159.36M}{EI}$$

$$\Delta H = \frac{-480.28V + 419.93H - 61.06M}{EI}$$

$$\Delta \theta = \frac{159.36V - 61.06H + 22.29M}{EI}$$

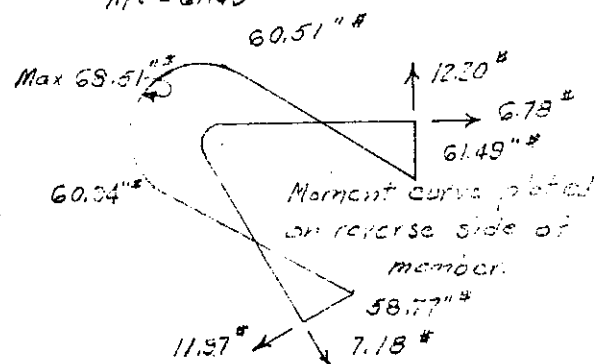
$$EI = 15930$$

Solving the equations

$$V = 12.20''$$

$$H = 6.78''$$

$$M = -61.49''$$



Moment Diagram For .01 Radian Rotation Inside plate

Max stress

$$[4.0/1875] + 68.51 \times 6/1875 = 11767 \text{ psi.}$$

Stresses are proportional to the rotation.

Case	Max. Rotation	Stress	Safety Factor
A1	.013321	15700 psi	2.10
A2	.011650	13700 psi	2.40
B1	.009843	11600 psi	2.84
B2	.011319	13300 psi	2.48
C1	.002200	10800 psi	3.05
C2	.009581	11300 psi	2.92

Station 44

Station 44 with hinged joints will be calculated without the stiff web from the Intrados corner to the Extrados side. In order to increase the factor at out. ly of the outside plate from the values shown in Modification #2, sheet II of LWI, Case A2 Westwind and B2 West wind, the outside skin plate will be made of clad steel plate with .05" stainless steel in lieu of 1/4" stainless steel plate as detailed. The clad steel will reduce the differential coefficient of expansion by 80% which in turn reduces the longitudinal temperature stresses. Horizontal stiffeners 12" and 27" each side of frame will be used on outside plates extending to the 1st longitudinal stiffeners from corners.

Summary of Stresses and Safety Factors (Hinged Joints)									
Case	Wind	Longitudinal Stresses			Transverse Stresses			Safety Factor Principal Stresses	Safety Factor Buckling
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total		
1/4" Outside Plate @ SEI									
A1	N	49 00 t	99 00 t	148 00 t	400 t	2400 t	2800 t	2.42	
	S	59 00 t	99 00 t	153 00 t	300 t	2400 t	2700 t	2.25	
	E	55 00 t	99 00 t	154 00 t	200 t	2400 t	2600 t	2.31	
	W	53 00 t	99 00 t	152 00 t	300 t	2400 t	2700 t	2.34	
A2	N	24 00 t	36 00 c	12 00 c	800 t	2200 c	14 00 c	—	—
	S	84 00 t	36 00 c	48 00 t	400 t	2200 c	18 00 c	5.58	—
	E	59 00 t	36 00 c	23 00 t	0	2200 c	22 00 c	8.46	—
	W	50 00 t	36 00 c	14 00 t	600 t	2200 c	16 00 c	—	—
B1	N	700 t	66 00 t	73 00 t	11 00 t	16 00 t	27 00 t	5.16	
	S	101 00 t	66 00 t	167 00 t	500 t	16 00 t	21 00 t	2.09	
	E	62 00 t	66 00 t	128 00 t	200 c	16 00 t	14 00 t	2.71	
	W	47 00 t	66 00 t	113 00 t	700 t	16 00 t	23 00 t	3.19	
B2	N	700 t	31 00 c	24 00 c	11 00 t	17 00 c	6 00 c	—	—
	S	101 00 t	31 00 c	70 00 t	500 t	17 00 c	12 00 c	4.80	—
	E	62 00 t	31 00 c	31 00 t	200 c	17 00 c	19 00 c	7.54	—
	W	47 00 t	31 00 c	12 00 t	700 t	17 00 c	10 00 c	—	—
C1	N	33 00 t	66 00 t	99 00 t	600 t	16 00 t	22 00 t	3.66	
	S	75 00 t	66 00 t	141 00 t	400 t	16 00 t	20 00 t	2.49	
	E	58 00 t	66 00 t	124 00 t	100 t	16 00 t	17 00 t	2.83	
	W	51 00 t	66 00 t	117 00 t	500 t	16 00 t	21 00 t	3.05	
C2	N	33 00 t	24 00 c	9 00 t	600 t	16 00 c	10 00 c	—	—
	S	75 00 t	24 00 c	51 00 t	400 t	16 00 c	12 00 c	5.69	—
	E	58 00 t	24 00 c	34 00 t	100 t	16 00 c	15 00 c	7.53	—
	W	51 00 t	24 00 c	27 00 t	500 t	16 00 c	11 00 c	9.74	—

Station 11

Summary of Stresses and Safety Factors Cont. (Hinged Joints)									
Case	Wind	Longitudinal Stresses			Transverse Stresses			Safety Factor	Safety Factor Backbone
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total	Principal Stresses	
1/4" Outside Plate @ ESI									
A1	N	4100 c	10500 t	6400 t	400 t	2400 t	2800 t	5.93	
	S	4600 c	10500 t	5900 t	400 t	2400 t	2800 t	6.45	
	E	3100 c	10500 t	7400 t	400 t	2400 t	2800 t	5.25	
	W	5600 c	10500 t	4900 t	500 t	2400 t	2900 t	7.73	
A2	N	3000 c	4400 c	7400 c	600 t	2200 c	1600 c	4.39	3.02
	S	5800 c	4400 c	10200 c	800 t	2200 c	1400 c	3.44	2.85
	E	3100 t	2200 c	1300 c	300 t	2200 c	1900 c	—	—
	W	11600 c	2200 c	13800 c	1000 t	2200 c	1200 c	2.13	1.85
B1	N	2200 c	7000 t	4800 t	700 t	1600 t	2300 t	7.33	
	S	6600 c	7000 t	200 t	1100 t	1600 t	2700 t	—	
	E	7300 t	7000 t	14300 t	300 t	1600 t	1900 t	2.45	
	W	15700 c	7000 t	8700 c	1300 t	1600 t	2900 t	3.15	3.45
B2	N	2200 c	3600 c	5800 c	700 t	1700 c	1000 c	6.14	5.04
	S	6600 c	3600 c	10200 c	1100 t	1700 c	600 c	3.32	2.92
	E	1300 t	2200 c	3700 t	300 t	1700 c	1400 c	7.22	—
	W	15700 c	2200 c	15900 c	1300 t	1700 c	400 c	1.72	1.55
C1	N	3100 c	7000 t	3600 t	500 t	1600 t	2100 t	—	
	S	5300 c	7000 t	1700 t	700 t	1600 t	2300 t	—	
	E	800 t	7000 t	7800 t	400 t	1600 t	2000 t	4.70	—
	W	9400 c	7000 t	2400 c	800 t	1600 t	2400 t	7.93	—
C2	N	5400 c	3000 c	6400 c	500 t	1600 c	1100 c	5.56	4.56
	S	5300 c	3000 c	3300 c	700 t	1600 c	900 c	4.73	3.54
	E	800 t	3000 c	2200 c	400 t	1600 c	1200 c	—	—
	W	9400 c	3000 c	12400 c	800 t	1600 c	800 c	2.74	2.39
1/4" Outside Plate @ EWI									
A1	N	4700 c	10600 t	5900 t	700 t	2300 t	3000 t	6.45	
	S	4300 c	10600 t	5300 t	800 t	2300 t	3100 t	7.15	
	E	3800 c	10600 t	6800 t	700 t	2300 t	3000 t	5.59	
	W	6200 c	10600 t	4400 t	900 t	2300 t	3200 t	8.21	
A2	N	3400 c	13600 c	13000 c	600 t	3300 c	2700 c	2.77	2.23
	S	6700 c	13600 c	16300 c	1400 t	3300 c	1900 c	2.13	1.81
	E	1000 t	13600 c	11600 c	600 t	3300 c	2700 c	3.13	2.49
	W	1800 c	13600 c	21400 c	1600 t	3300 c	1700 c	1.60	1.33
B1	N	2400 c	1500 t	4600 t	500 t	1600 t	2100 t	5.27	
	S	7700 c	1500 t	700 c	1800 t	1600 t	3400 t	8.68	—
	E	5900 t	1500 t	11200 t	600 t	1600 t	2200 t	3.76	
	W	15700 c	1500 t	8700 c	2100 t	1600 t	3700 t	3.77	3.45

Station 44

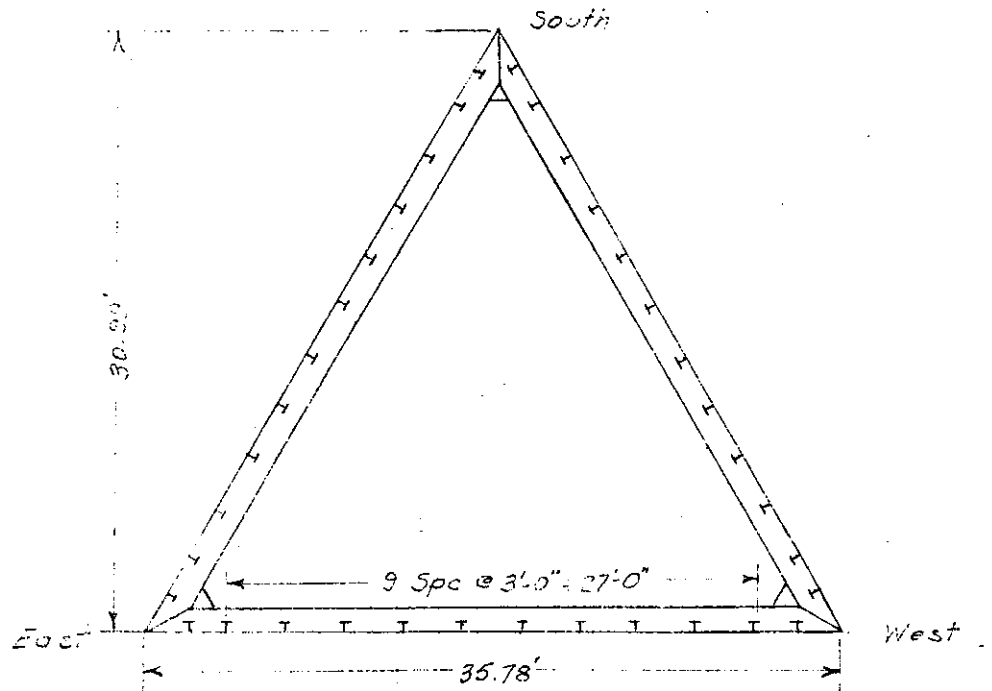
Summary of Stresses and Safety Factors Cont. (Hinged Joints)									
Case	Dir.	Longitudinal Stresses			Transverse Stresses			Safety Factor	Safety
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total	Principal Stresses	Factor Buckling
1/4" Outside Plate @ EWI									
B2	N	2400 c	8400 c	10800 c	500 t	2700 c	2200 c	3.33	2.69
	S	7700 c	8400 c	16100 c	1800 t	2700 c	900 c	2.10	1.82
	E	5900 t	8400 c	2500 c	600 t	2700 c	2100 c	—	—
	W	15700 c	8400 c	24100 c	2100 t	2700 c	600 c	1.32	1.24
C1	N	3900 c	7000 t	3100 t	600 t	1600 t	2200 t	—	—
	S	6200 c	7000 t	800 t	1200 t	1600 t	2800 t	—	—
	E	200 c	7000 t	6800 t	600 t	1600 t	2200 t	5.40	—
	W	9700 c	7000 t	2700 c	1300 t	1600 t	2900 t	6.80	—
C2	N	3900 c	8400 c	12300 c	600 t	2700 c	2100 c	2.39	2.37
	S	6200 c	8400 c	14600 c	1200 t	2700 c	1500 c	2.37	2.02
	E	200 c	8400 c	8600 c	600 t	2700 c	2100 c	4.24	3.35
	W	9700 c	8400 c	18100 c	1300 t	2700 c	1400 c	1.39	1.64
1/4" Outside Plate @ EEW									
A1	N	4700 c	7700 t	3000 t	1400 t	200 t	1600 t	—	—
	S	5300 c	7700 t	3400 t	1800 t	200 t	2000 t	—	—
	E	5000 c	7700 t	2700 t	1700 t	200 t	1900 t	—	—
	W	5000 c	7700 t	2700 t	1700 t	200 t	1900 t	—	—
A2	N	3400 c	7100 c	10500 c	500 t	300 c	200 t	3.11	2.36
	S	6700 c	7100 c	13800 c	3100 t	300 c	2800 t	2.07	2.17
	E	5000 c	7100 c	12100 c	2500 t	300 c	2200 t	2.47	2.48
	W	5000 c	7100 c	12100 c	2500 t	300 c	2200 t	2.47	2.48
B1	N	2400 c	5100 t	2700 t	100 c	200 t	100 t	—	—
	S	7700 c	5100 t	2600 c	4000 t	200 t	4200 t	5.55	—
	E	4900 c	5100 t	200 t	3000 t	200 t	3200 t	—	—
	W	4900 c	5100 t	200 t	3000 t	200 t	3200 t	—	—
B2	N	2400 c	6200 c	8400 c	100 c	300 c	400 c	4.02	3.55
	S	7700 c	6200 c	13900 c	4000 t	300 c	3700 t	2.05	2.16
	E	4900 c	6200 c	11100 c	3000 t	300 c	2700 t	2.60	2.71
	W	4900 c	6200 c	11100 c	3000 t	300 c	2700 t	2.60	2.71
C1	N	3900 c	5100 t	1200 t	800 t	200 t	1000 t	—	—
	S	6200 c	5100 t	1100 c	2600 t	200 t	2800 t	2.47	—
	E	5000 c	5100 t	100 t	2200 t	200 t	2400 t	—	—
	W	5000 c	5100 t	100 t	2200 t	200 t	2400 t	—	—
C2	N	3900 c	6200 c	10100 c	800 t	300 c	500 t	3.12	2.97
	S	6200 c	6200 c	12400 c	2600 t	300 c	2300 t	2.40	2.42
	E	5000 c	6200 c	11200 c	2200 t	300 c	1900 t	2.60	2.68
	W	5000 c	6200 c	11200 c	2200 t	300 c	1900 t	2.60	2.68

Station 44

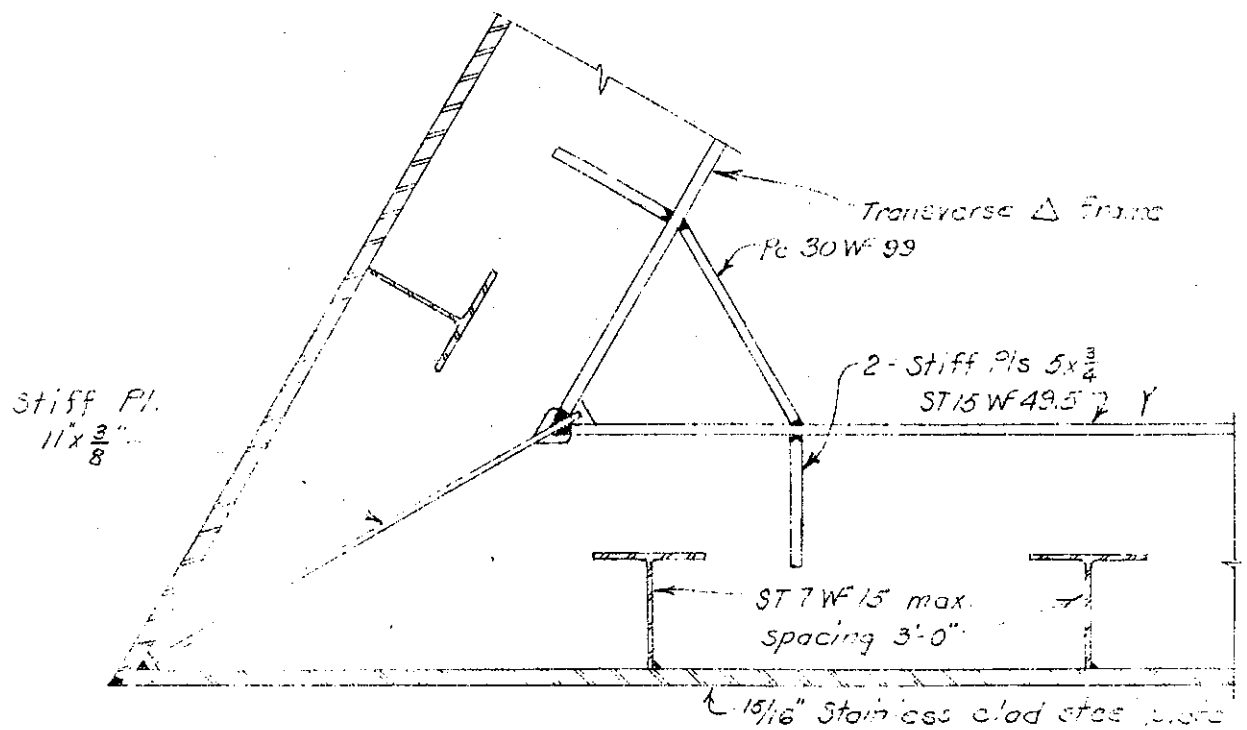
Summary of Stresses and Safety Factors Cent. (Hinged Joints)									
Case	Wind	Longitudinal Stresses			Transverse Stresses			Safety Factor Principal Stresses	Safety Factor Buckling
		Bending + Axial	Temp	Total	Bending + Axial	Temp	Total		
3/8" Inside Plate @ E & W									
A1	N	44 00 c	52 00 c	96 00 c	900 c	200 c	700 c	3.55	2.52
	S	49 00 c	52 00 c	101 00 c	13 00 c	200 c	11 00 c	3.43	2.33
	E	45 00 c	52 00 c	98 00 c	12 00 c	200 c	10 00 c	3.53	2.47
	W	48 00 c	52 00 c	98 00 c	12 00 c	200 c	10 00 c	3.53	2.42
A2	N	32 00 c	45 00 t	13 00 t	300 c	300 c	600 c	—	—
	S	62 00 c	45 00 t	17 00 c	22 00 c	300 c	25 00 c	—	2.17
	E	46 00 c	45 00 t	1 00 c	15 00 c	300 c	21 00 c	—	4.23
	W	46 00 c	45 00 t	1 00 c	15 00 c	300 c	21 00 c	—	4.05
B1	N	28 00 c	34 00 c	38 00 c	100 c	200 c	300 c	5.54	4.41
	S	21 00 c	35 00 c	12 00 c	28 00 c	200 c	25 00 c	3.41	1.96
	E	25 00 c	35 00 c	60 00 c	22 00 c	200 c	20 00 c	4.57	2.39
	W	42 00 c	35 00 c	80 00 c	22 00 c	200 c	20 00 c	4.57	2.57
B2	N	28 00 c	41 00 t	13 00 t	100 t	300 c	200 c	—	—
	S	57 00 c	41 00 t	30 00 c	28 00 c	300 c	31 00 c	—	2.77
	E	45 00 c	41 00 t	4 00 c	22 00 c	300 c	25 00 c	—	3.84
	W	46 00 c	41 00 t	4 00 c	22 00 c	300 c	25 00 c	—	3.57
C1	N	35 00 c	35 00 c	71 00 c	600 c	200 c	400 c	4.77	2.44
	S	57 00 c	35 00 c	92 00 c	13 00 c	200 c	16 00 c	3.27	2.43
	E	63 00 c	35 00 c	81 00 c	12 00 c	200 c	14 00 c	4.22	2.74
	W	63 00 c	35 00 c	81 00 c	12 00 c	200 c	14 00 c	4.22	2.74
C2	N	36 00 c	41 00 c	58 00 c	600 c	700 c	900 c	—	—
	S	57 00 c	41 00 t	16 00 c	18 00 c	300 c	21 00 c	—	4.12
	E	46 00 c	41 00 t	5 00 c	16 00 c	300 c	19 00 c	—	4.51
	W	46 00 c	41 00 t	5 00 c	16 00 c	300 c	19 00 c	—	4.54

Station 37

ALTERNATE DESIGN - SINGLE PLATE WALL



PLAN OF FRAME



PLAN OF JOINT

Station 31

Properties of steel

$$Area = \frac{3(35.78)(.9375)}{12} + \frac{36(4.41)}{144} = 9.49 \text{ in}^2$$

(Original design $A = 9.30 \text{ in}^2$)

$$I_x \approx I_y = \frac{2(1.9375)(10.39)^3}{(8.6603)(12)(12)} = 447.48$$

$$2 \text{ Pls } 5.5906 \times 5.145^2 = 149.14$$

$$1 \text{ Pl } 2.7953 \times 10.2909^2 = 296.03$$

$$12 \text{ Ts } 1.3675 \times 2.2069^2 = 35.34$$

$$24 \text{ Ts } 1.7259 \times 5.145^2 = 19.61$$

$$947.6 \text{ in}^4$$

(Original design $I = 1086$)Torsional stiffness

$$J = \frac{.25(1.9375)^3(.9375)}{12} = 894.64 \text{ in}^4$$

(Original design $J = 534 \text{ in}^4$)Plate Stiffness

See page 331 "Theory of Elastic Stability" by Timoshenko.

$$\frac{\sigma_{cr}}{\sqrt{E}} = \frac{K \pi^2 E}{12(1-\nu^2)} \left(\frac{h}{b}\right)^2, K=4$$

If $\sigma_{cr} = 31,000 \text{ psi}$, then

$$\sigma_{cr}/\sqrt{E} = 56,000$$

$$56,000 = \frac{4(31,000)^2(1.9375)^2}{10.92} \left(\frac{h}{b}\right)^2$$

$$\frac{h}{b} = \sqrt{.000341} \quad \frac{h}{b} = .0231$$

$$\therefore h = 18/16 \quad b = 40"$$

Section at Station 37

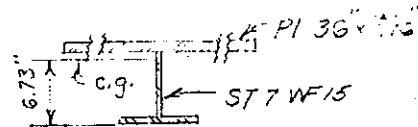
Width outside = 35.78" or 429.36"

Assume width of unsupported side = 405"

Assume 2 stiffeners @ 36" ctrs

Assume 2 frames @ 144" ctrs

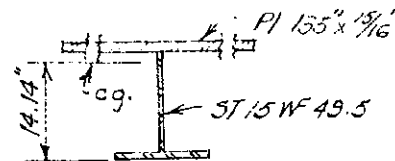
Assume effective width of

transverse plate $1/3(405) = 135"$ 

$$A = 38.16 \text{ in}^2$$

$$I = 153.09 \text{ in}^4$$

$$I \text{ per inch width} = 4.2521 \text{ in}^4$$

Longitudinal Stiffeners
(@ 36" ctrs)

$$A = 141.12 \text{ in}^2$$

$$I = 1959.01 \text{ in}^4$$

$$I \text{ per inch width} = 13.6042$$

Transverse Stiffeners

(@ 144" ctrs)

(Transverse Δ Frames)

See page 331 "Theory of Elastic Stability" by Timoshenko.

$$\frac{\sigma_{cr}}{\sqrt{E}} = \frac{\pi^2}{b^2 h} \left(D_1 \frac{b^2}{a^2} + 2D_3 + D_2 \frac{a^2}{b^2} \right)$$

$$D_1 = \frac{E I_x}{1 - \nu_x \nu_y}, \nu_x = \nu_y = .30$$

$$D_1 = \frac{E(4.2521)}{.91} = 4.6726 E$$

$$D_2 = E I_y / .91 = 13.6042 E / .91 = 14.9497 E$$

$$D_3 = \frac{1}{2} \nu (D_2 + D_1) + 2 G I_{xy}$$

 I_{xy} is small and it will be slightly on the safe side to neglect it.Assume $I_{xy} = 0$

$$D_3 = .15(4.6726 + 14.9497) E = 2.9433 E$$

$$\frac{\sigma_{cr}}{\sqrt{E}} = \frac{\pi^2 \cdot 29,400^2}{405^2(1.9375)} \left\{ 4.6726 \left(\frac{b}{a}\right)^2 + 5.8966 + 14.9497 \left(\frac{a}{b}\right)^2 \right\}$$

$$= 1861.31 \left\{ 4.67 \left(\frac{b}{a}\right)^2 + 5.89 + 14.95 \left(\frac{a}{b}\right)^2 \right\}$$

$$\frac{a}{b} = \sqrt[4]{\frac{D_1}{D_2}} = \sqrt[4]{\frac{4.6726}{14.9497}} = .7477 \quad \lambda = 3.24$$

It is logical to assume that nodal lines will occur at transverse frames, i. use a = some multiple of 144", in this case 288"

$$a = 288" \quad b/b = 1.4063 \quad a/b = 1.7111$$

$$\frac{\sigma_{cr}}{\sqrt{E}} = 42230 \quad \sigma_{cr} = 29,640 \text{ psi}$$

See Table 27, page 343 "Buckling Strength of Metal Structures" by Bleich.

From page 382 Timoshenko

$$\begin{aligned} \frac{\sigma_{cr}}{\sqrt{E}} &= \frac{2\pi^2}{b^2h} (\sqrt{D_1 D_2} + D_3) \dots\dots\dots (233) \\ &= 2(1861.31)(7.467(14.95) + 2.94) \\ &= 42070 \approx 42230 \quad \text{OK} \end{aligned}$$

Section at Station 44

Width outside = 48.16" or 48.92"

Assume width of unsupported side = 457"

$$a = 288 \quad b/b = 1.5868 \quad a/b = 1.6302$$

$$\frac{\sigma_{cr}}{\sqrt{E}} = 34,483 \quad \sigma_{cr} = 28,250$$

Plate Stiffeners

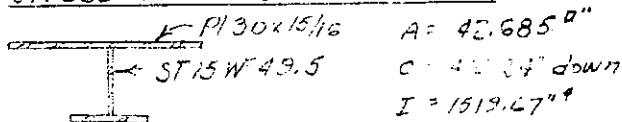
From sheet I-47

Using $\sigma_{cr}/\sqrt{E} = 34,483$

$$\frac{h}{b} = .0181 \quad \text{if } h = .9375"$$

$b = 51"$ i. 36" spacing is OK.

Stress in Transverse Frame



$$\text{Max } M = 32.6 \text{ k} \quad N = 14.5 \text{ k}$$

$$P/A = 341 \text{ psi} \text{ comp. } M/S = 1631 \text{ psi c. o.s.}$$

$$2961 \text{ psi t. i.s.}$$

$$\text{for } 140 \text{ ksi } 1.5 \text{ ST}$$

$$\text{for } 140 \text{ ksi } 1.5 \text{ ST}$$

Stability of Longitudinal Stiffeners

See Bleich page 381

$$\text{Web } \frac{\sigma_{cr}}{\sqrt{E}} = \frac{\pi^2 29,406}{10.92} \left(\frac{.27}{6.7385} \right)^2 K$$

$$c/b = 3.3665/6.7385 = .4996$$

$$t_c/t = .383/.27 = 1.4185$$

$$K = 4.0$$

$$\sigma_{cr}/\sqrt{E} = 163,300 \text{ psi} \quad \text{OK}$$

$$\begin{aligned} \text{Flange } \frac{\sigma_{cr}}{\sqrt{E}} &= \frac{.425 \cdot \pi^2 29,406}{10.92} \left(\frac{.383}{3.3665} \right)^2 \\ &= 144,200 \text{ psi} \quad \text{OK} \end{aligned}$$

Stability of Transverse Stiffeners

$$\text{Web } \frac{\sigma_{cr}}{\sqrt{E}} = \frac{\pi^2 29,406}{10.92} \left(\frac{.522}{14.985} \right)^2 K$$

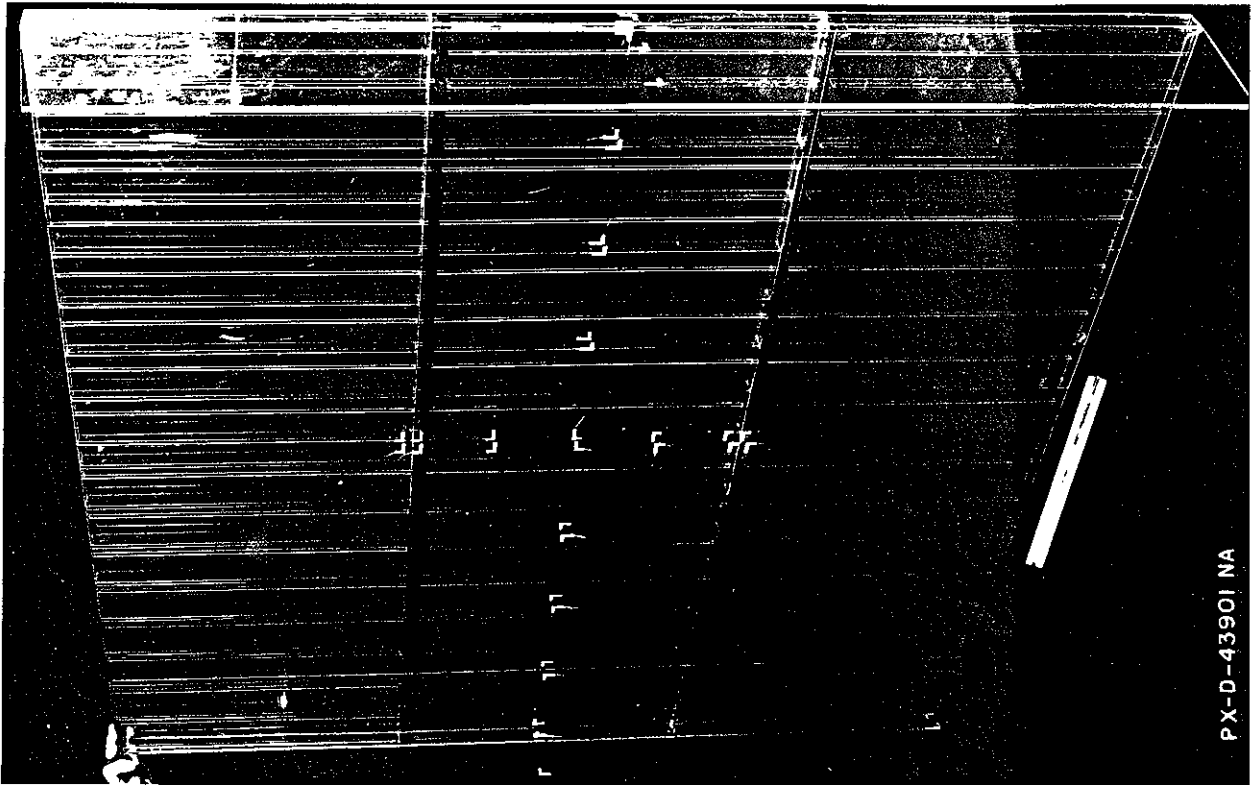
$$c/b = 5.229/14.985 = .3490$$

$$t_c/t = .670/.522 = 1.2835$$

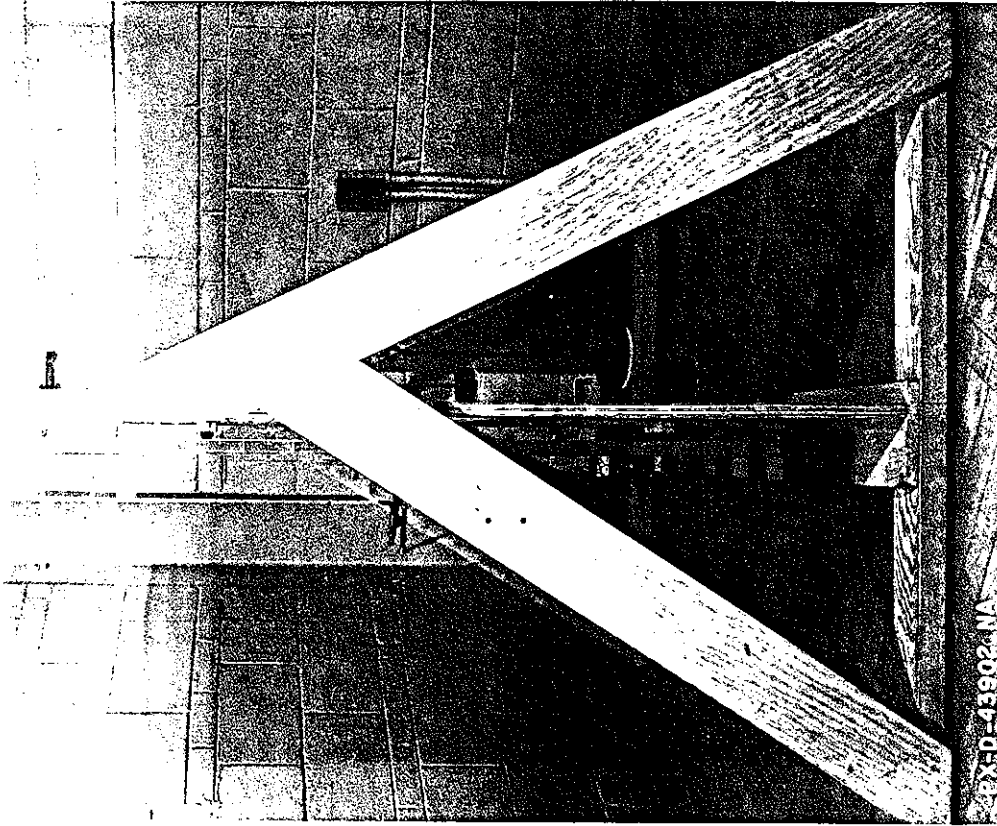
$$K = 4.5527$$

$$\sigma_{cr}/\sqrt{E} = 155,000 \text{ psi} \quad \text{OK}$$

$$\begin{aligned} \text{Flange } \frac{\sigma_{cr}}{\sqrt{E}} &= \frac{.425 \cdot \pi^2 29,406}{10.92} \left(\frac{.522}{14.985} \right)^2 \\ &= 132,900 \text{ psi} \quad \text{OK} \end{aligned}$$

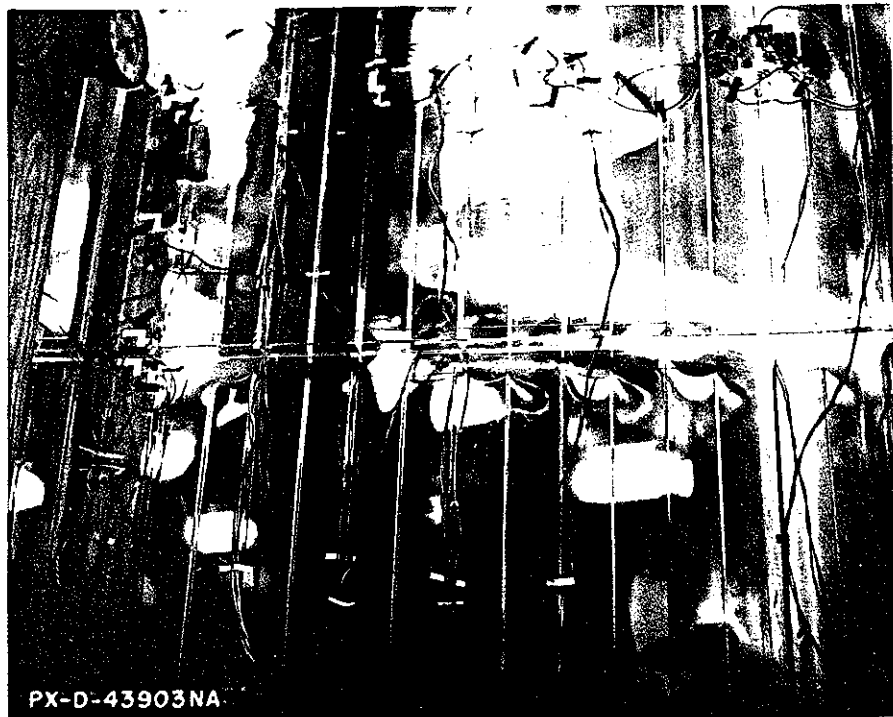


Model

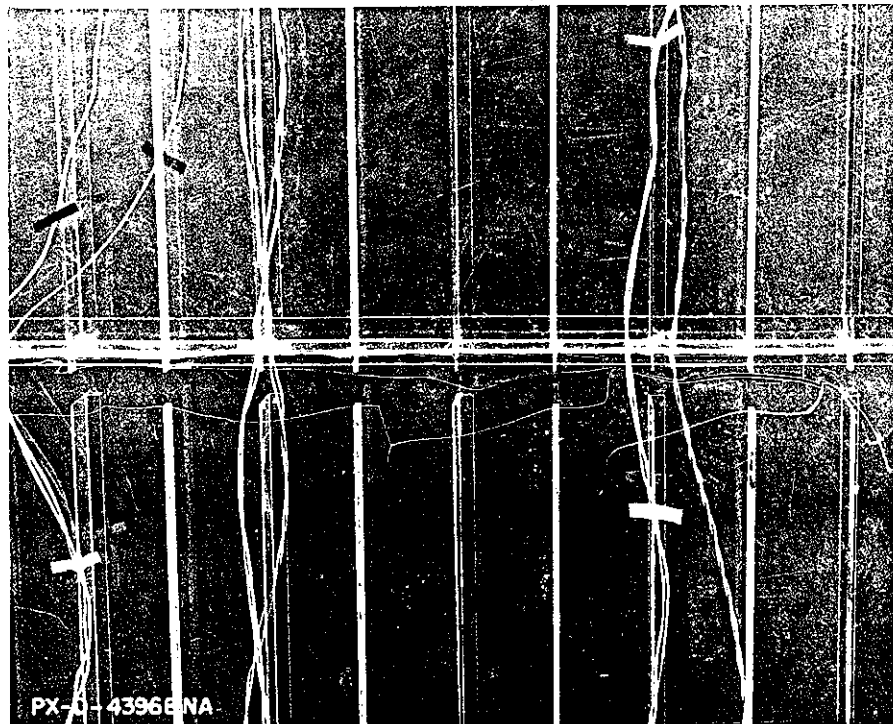


Side View
Deflection of Model for Determination of Elastic Constants

Figure V-1



Reflections Showing Exterior Skin Plate Distortion



Failure of Exterior Skin Plate

Figure Y-2

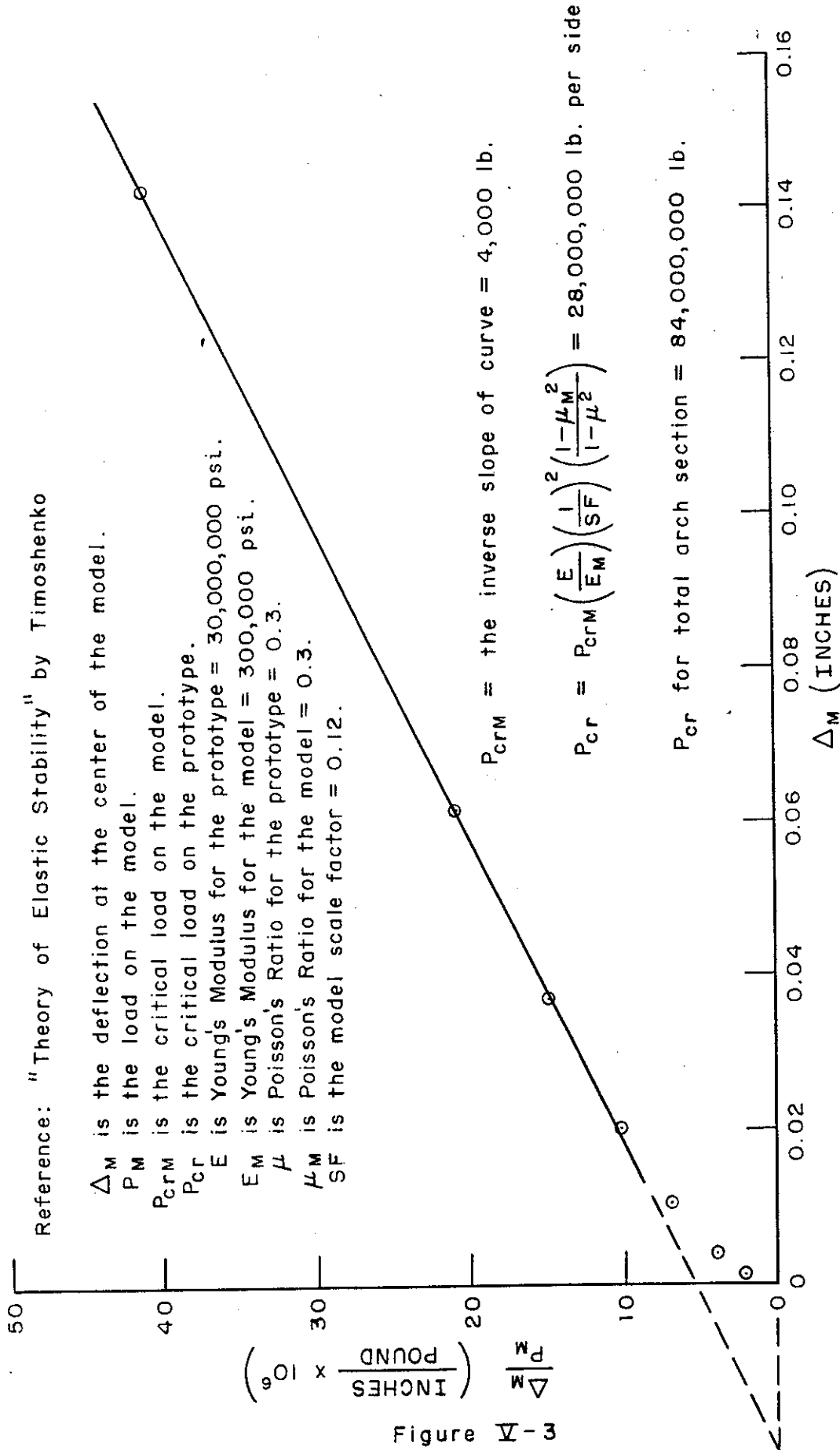


Figure V-3

JEFFERSON MEMORIAL ARCH
 MODEL STUDY
 CRITICAL LOAD FROM PLATE DEFLECTION

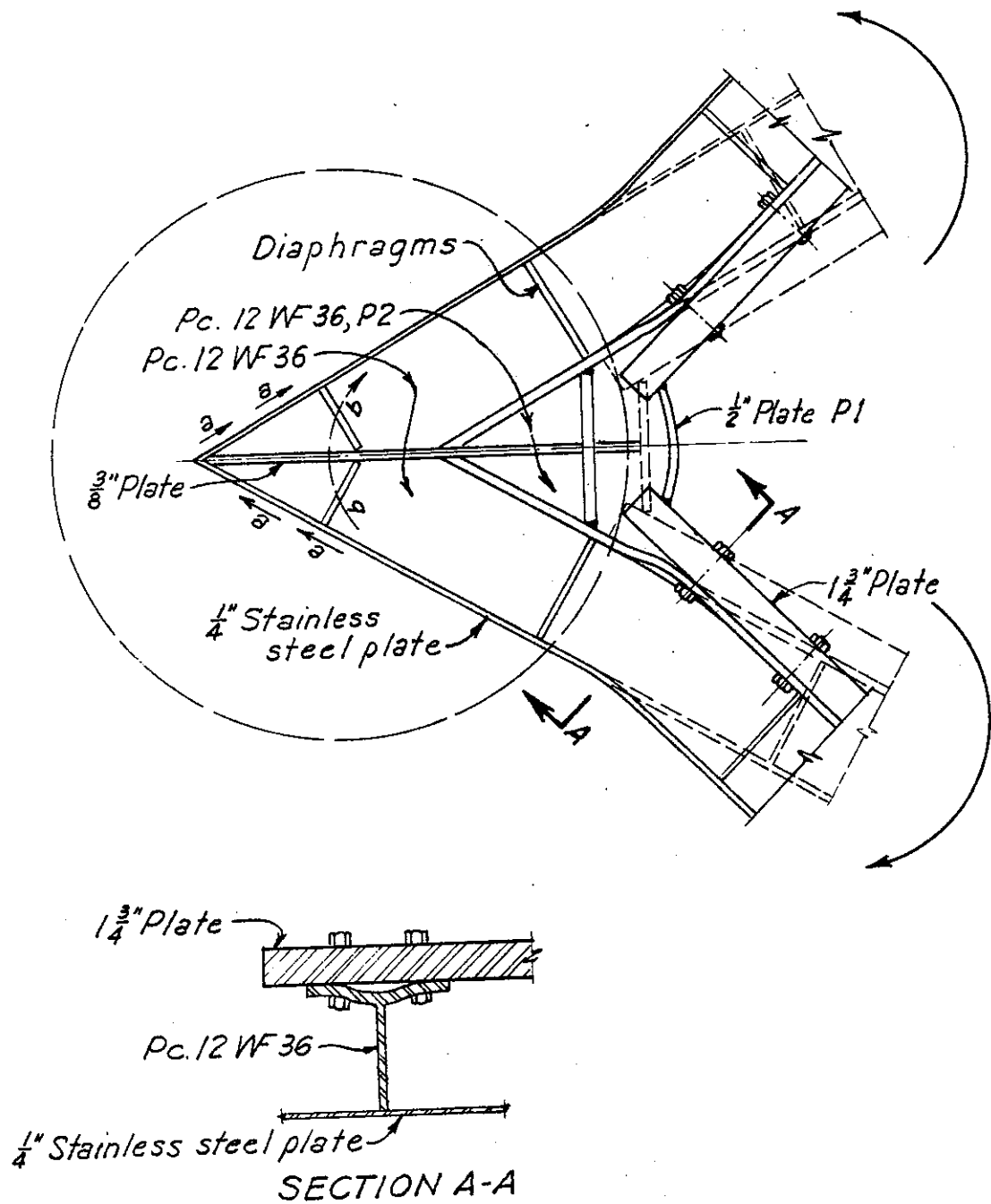
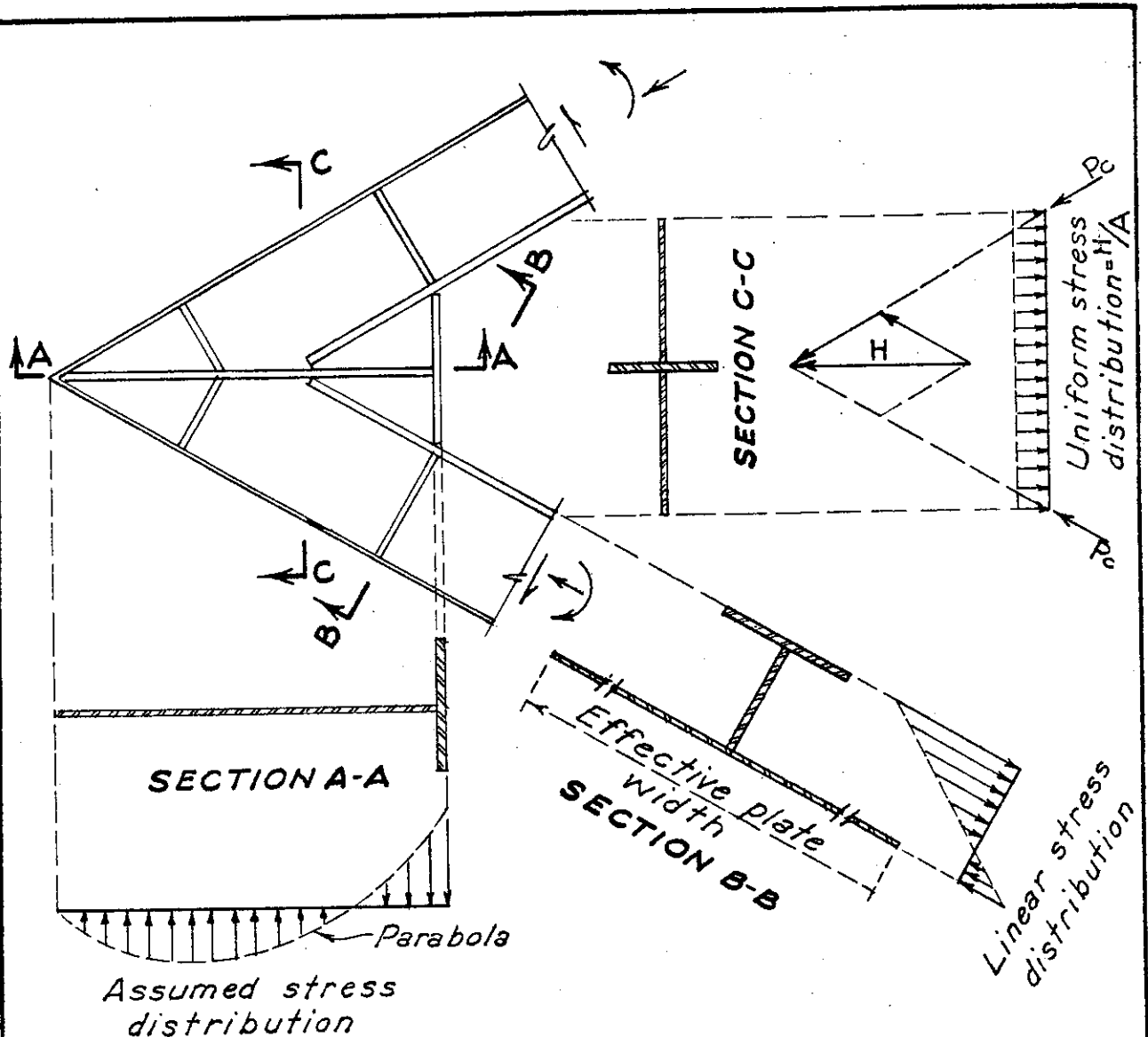


Figure V-4
 RIGID CORNER
 WITH APPLIED MOMENT



NOTE

The forces P_c represent the total stress in the outside skin plates at Section C-C. To evaluate them, first determine the stress in these plates at Section B-B then reduce the stress in linear proportion from B to C, assuming the stress goes to zero at the apex of the corner.

Figure V-5
RIGID CORNER
ASSUMED STRESS DISTRIBUTION

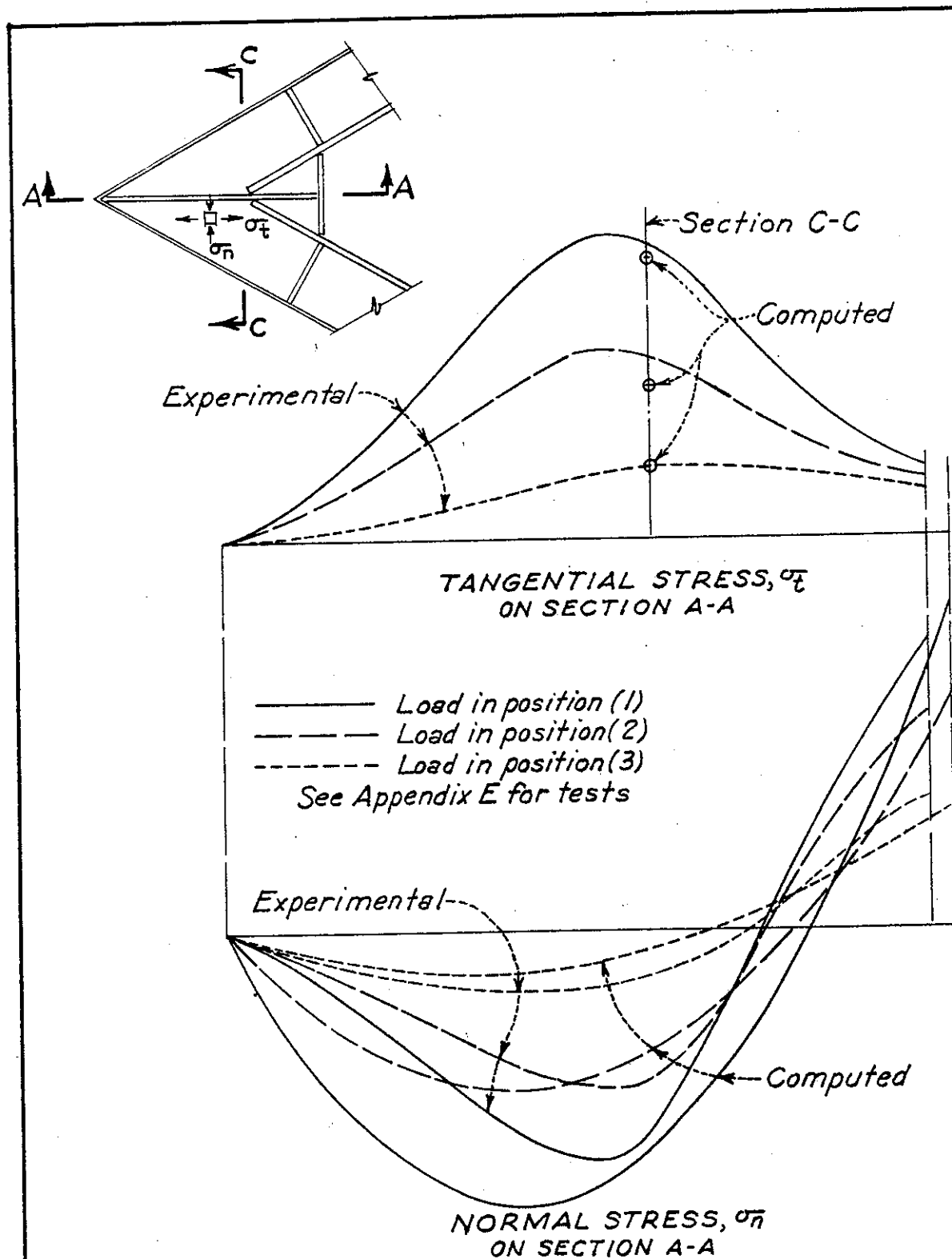
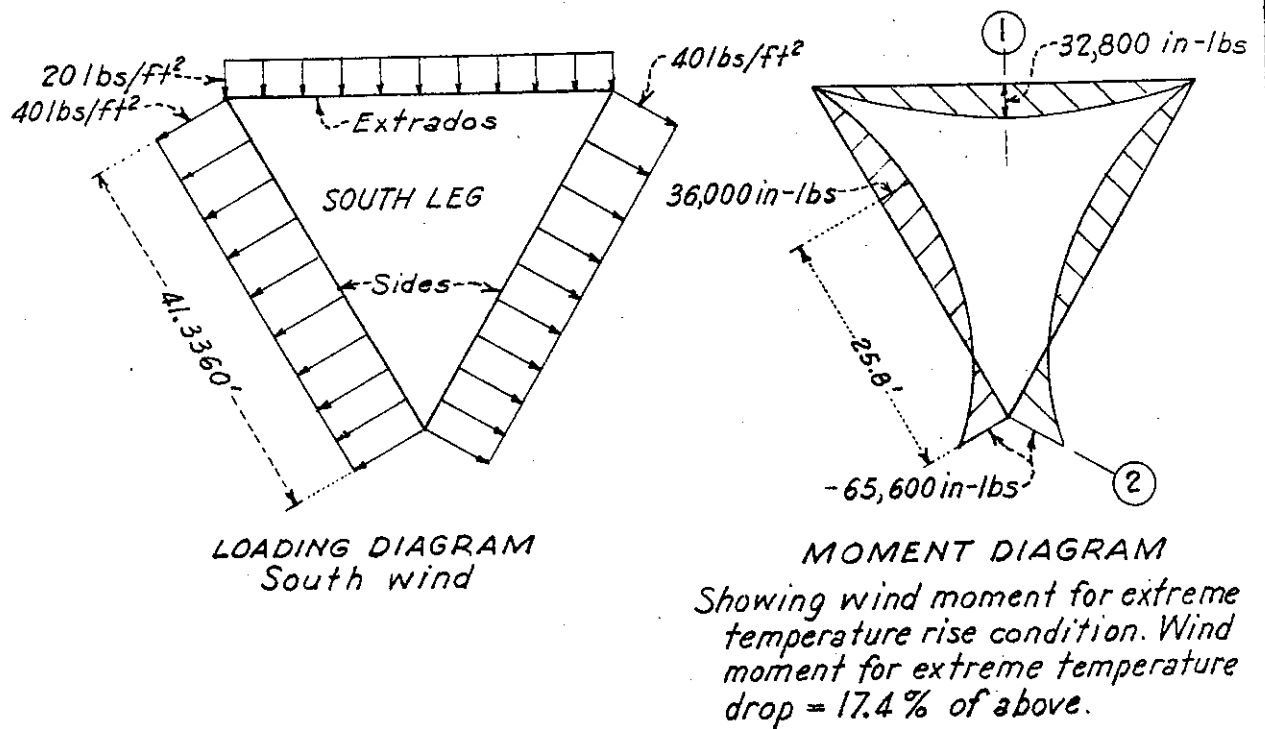


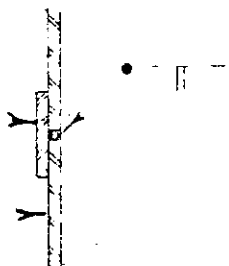
Figure V-6
RIGID CORNER
COMPARISON OF EXPERIMENTAL
AND COMPUTED STRESSES



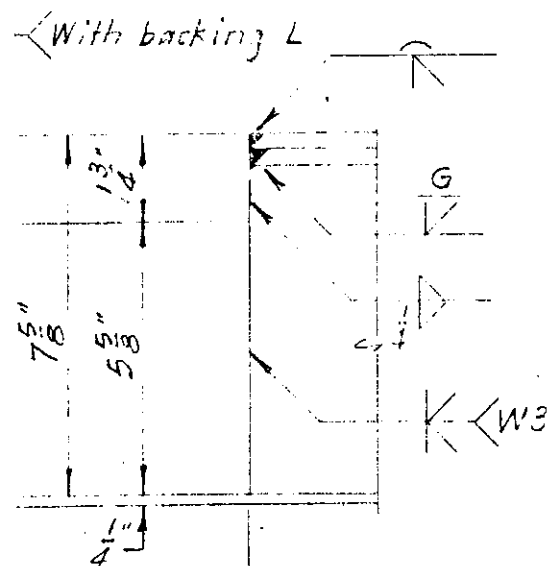
Section	Loading Case A-1 Extreme temp. drop			Loading Case A-2 Extreme temp. rise			
	Temp.	Wind	Total	Temp.	Shrink.	Wind	Total
①							
Outer R	+9391psi		+9391psi	-416	+134	-60	-342psi
Outer conc.							
Conc. 2" in	+281	-14	+267				
Inner conc.	-447	+10	-437	+373	+68	+46	+487
Inner R	-1016	+64	-952	+912	-4054	+291	-2851
②							
Outer R	+8626		+8626	-88	+57	-101	-132
Outer conc.							
Conc. 2" in	+652	-44	+608				
Inner conc.	-830	+44	-786	+176	+262	+48	+486
Inner R				-337	-1450	+310	-1477

Figure V-7
COMBINED TRANSVERSE STRESSES
STATION 46

continuous
shoring 1/2" x 1/2"



SEC. A-A
HORIZONTAL SPlice

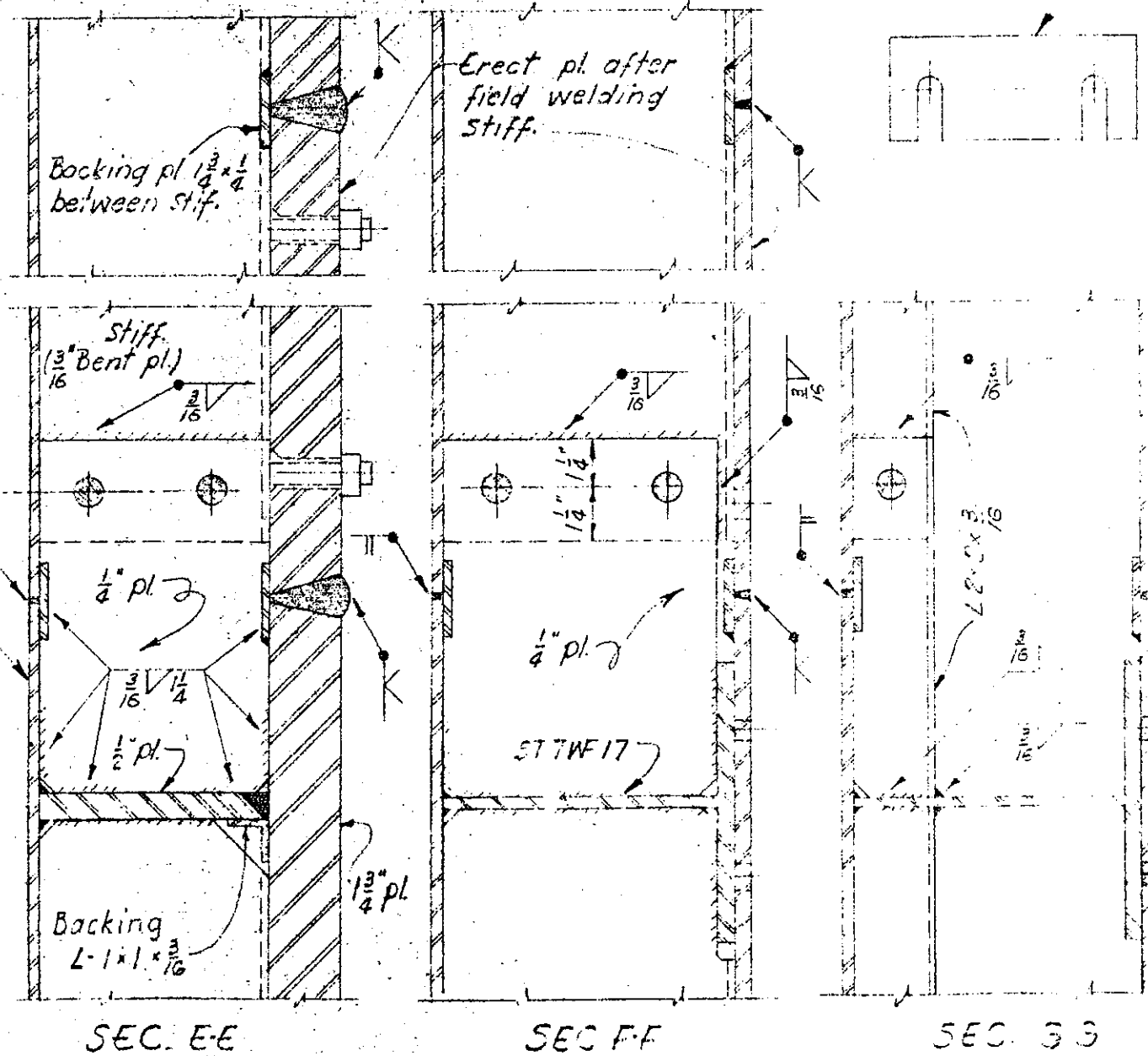
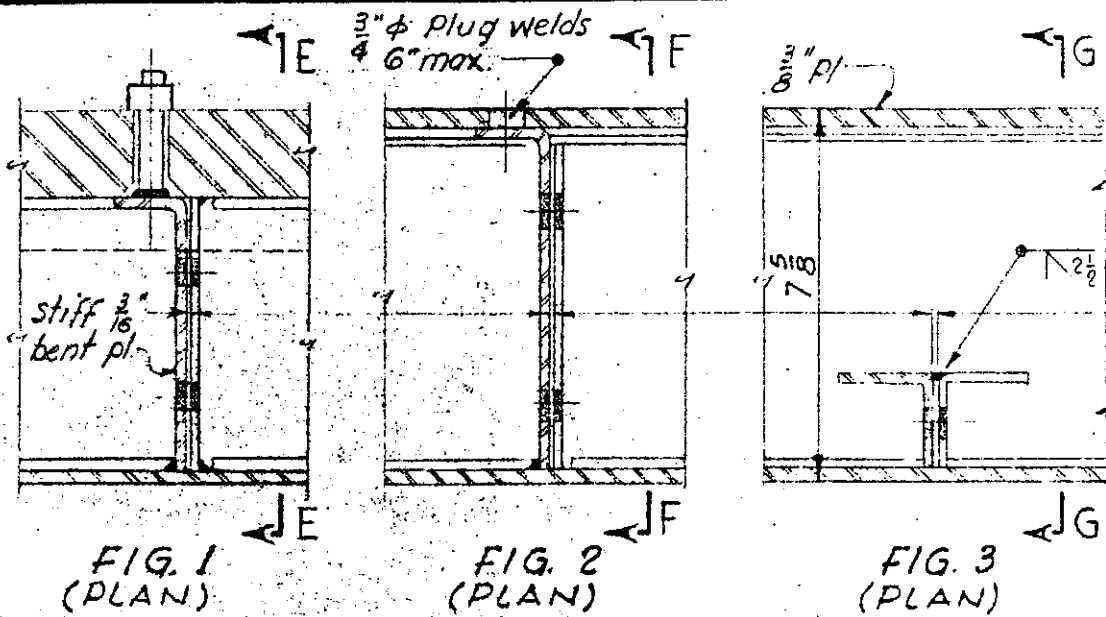


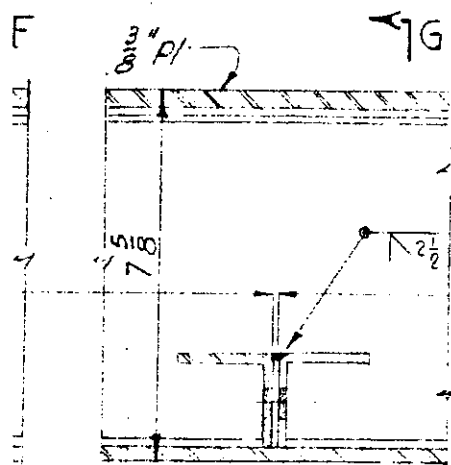
* Pin shall be comparable
to ASTM. A441 steel, with
46,000 psi specified yield
point.



WELD W3

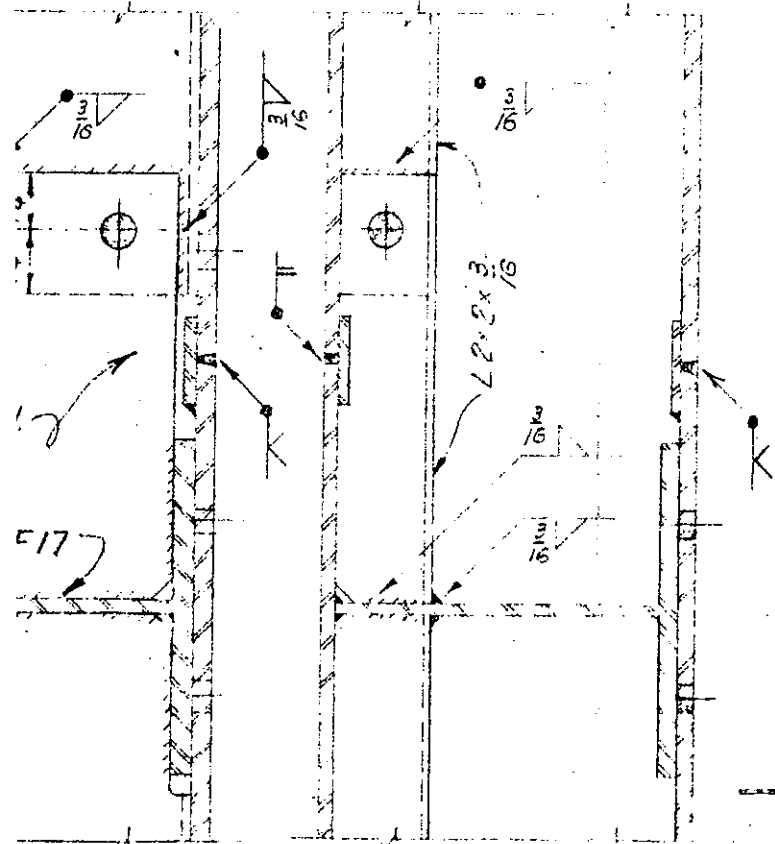
11-2-64 D. CRJ	REVISED
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION JNEM GATEWAY ARCH TRIANGULAR FRAME HINGE DETAIL. STATION 15	
DRAWN <u>JGA</u>	SUBMITTED <u>HC Clander</u>
TRACED <u>LFH</u>	RECOMMENDED
CHECKED <u>mf</u>	APPROVED
DENVER, COLO. JUNE 4, 1964 X-OA-D-903	

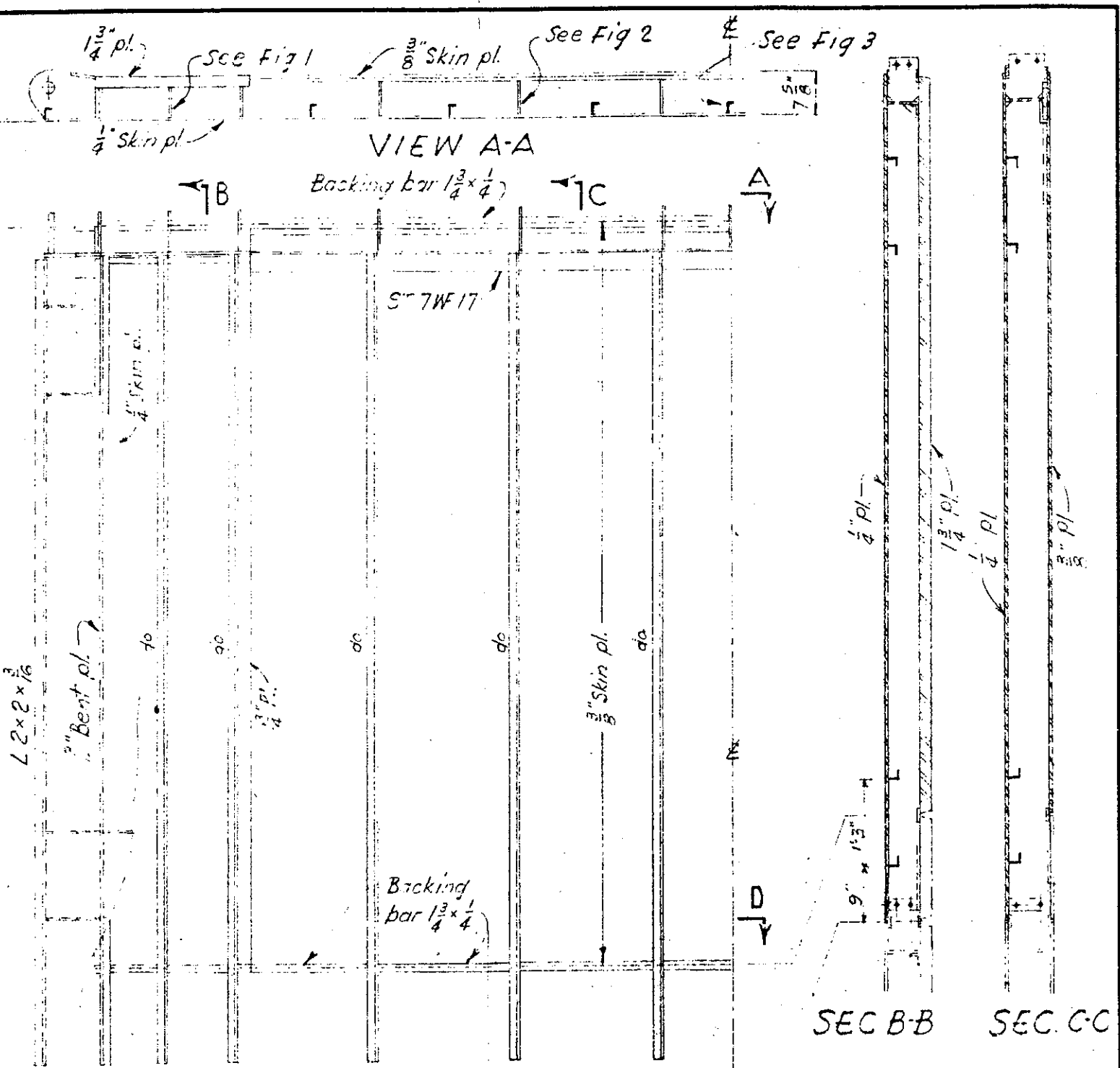




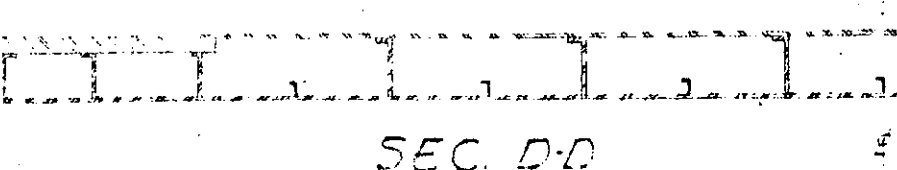
Offset stiff for erection clearance. If clearance exceeds 1/8", use shims.

after welding



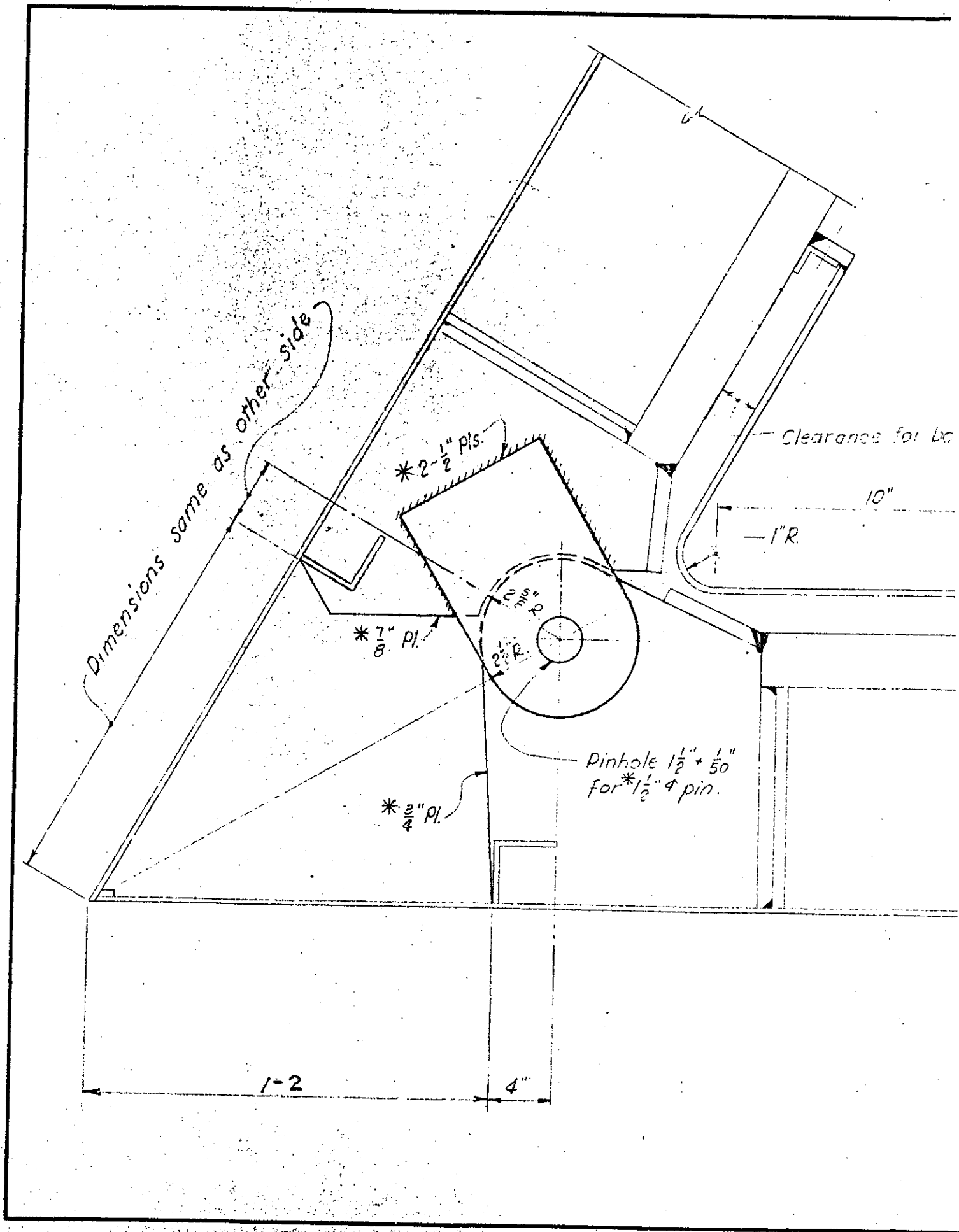


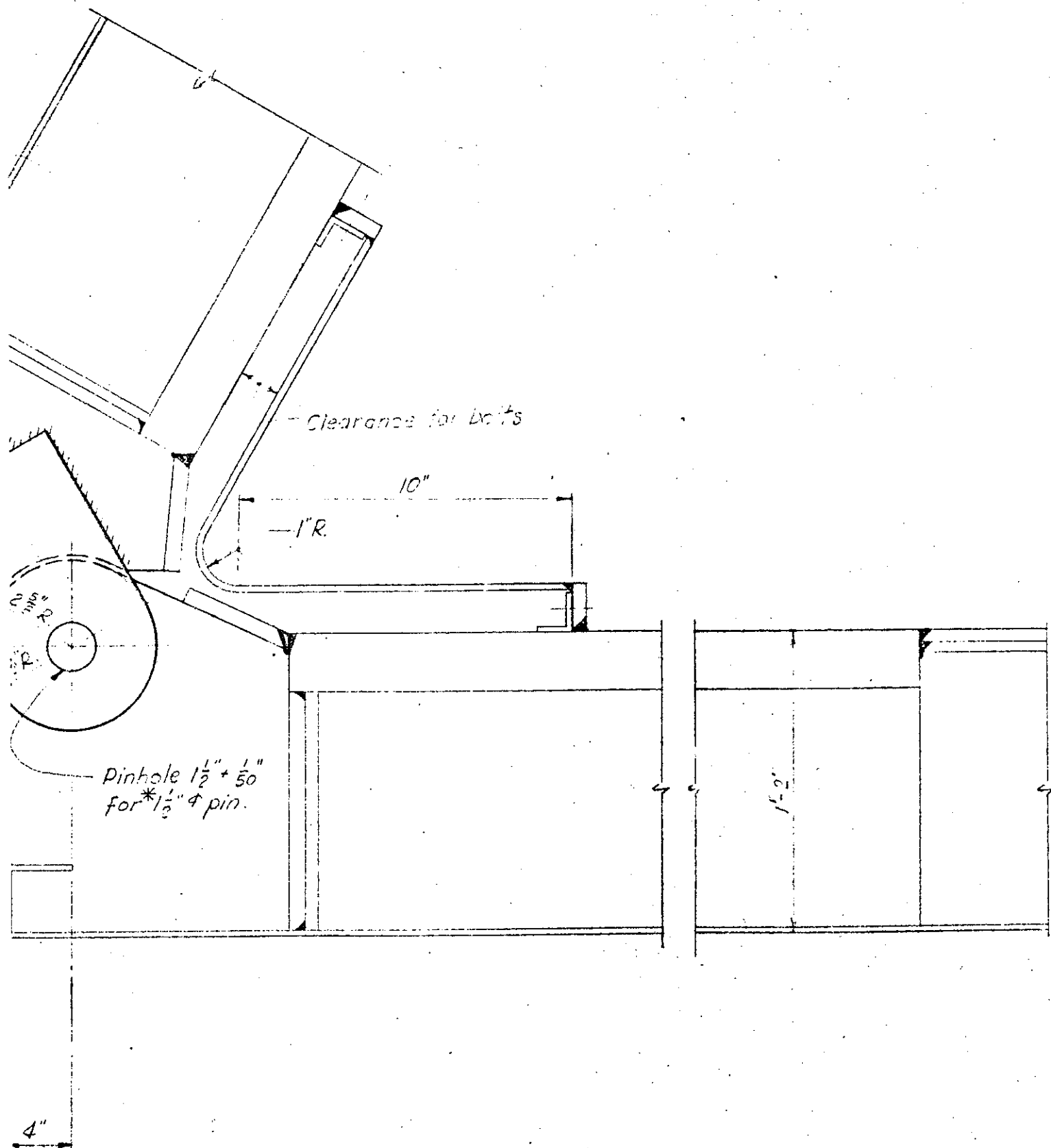
ELEVATION
(Outside skin pl. Removed)



SEC. D-D

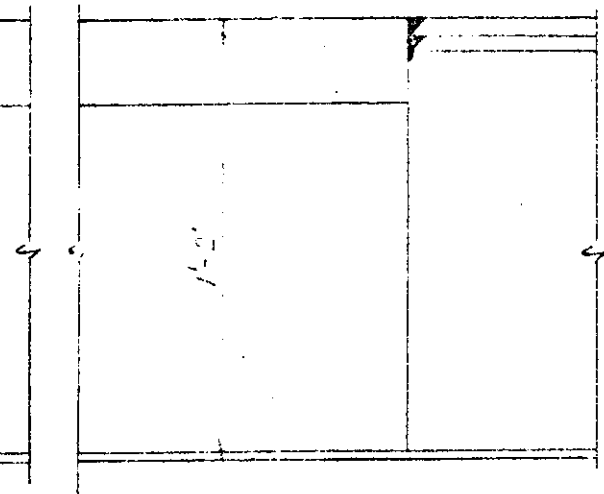
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	
JNEM GATEWAY ARCH LONGITUDINAL DIAPHRAGMS & STIFFENERS FIELD CONNECTION DETAILS STA. 15	
DRAWN <u>JGA</u>	SUBMITTED <u>H.C. Clendenen</u>
TRACED <u>LEH</u>	RECOMMENDED
CHECKED <u>ca</u>	APPROVED
DENVER COLORADO JUNE 4, 1934	
X-2A-D-994	




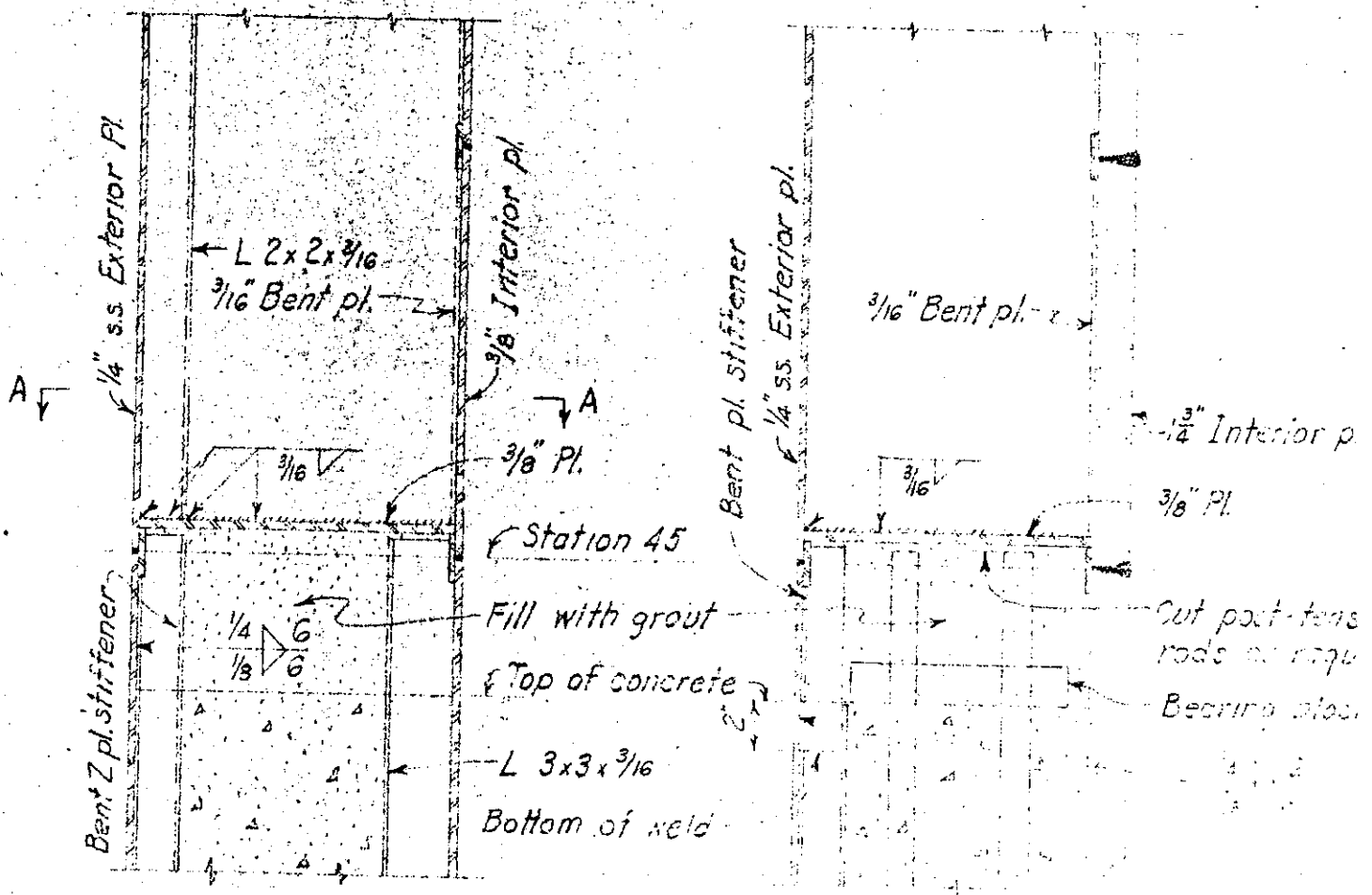


NOTES

Details not shown are similar to details shown on Dwg X-0A-D-903.
 * Indicates material shall be comparable to ASTM A441 steel, with 46,000 psi specified yield point, except that where clearances and design of details permit, 2" ϕ pins, and plates, of ASTM A36 steel may be used.

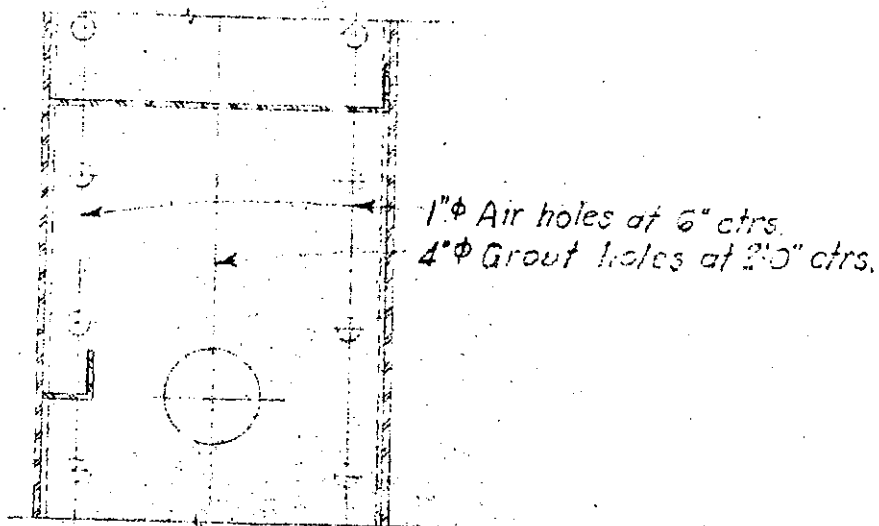


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 ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	
JNEM GATEWAY ARCH TRIANGULAR FRAME HINGE DETAIL STATION 44	
DRAWN <u>V.G.A.</u> SUBMITTED <u>H. Colander</u> TRACED <u>L.F.M.</u> RECOMMENDED _____ CHECKED <u>CA</u> APPROVED _____	
DENVER, COLO. JUNE 4, 1964 X-0A-D-905	



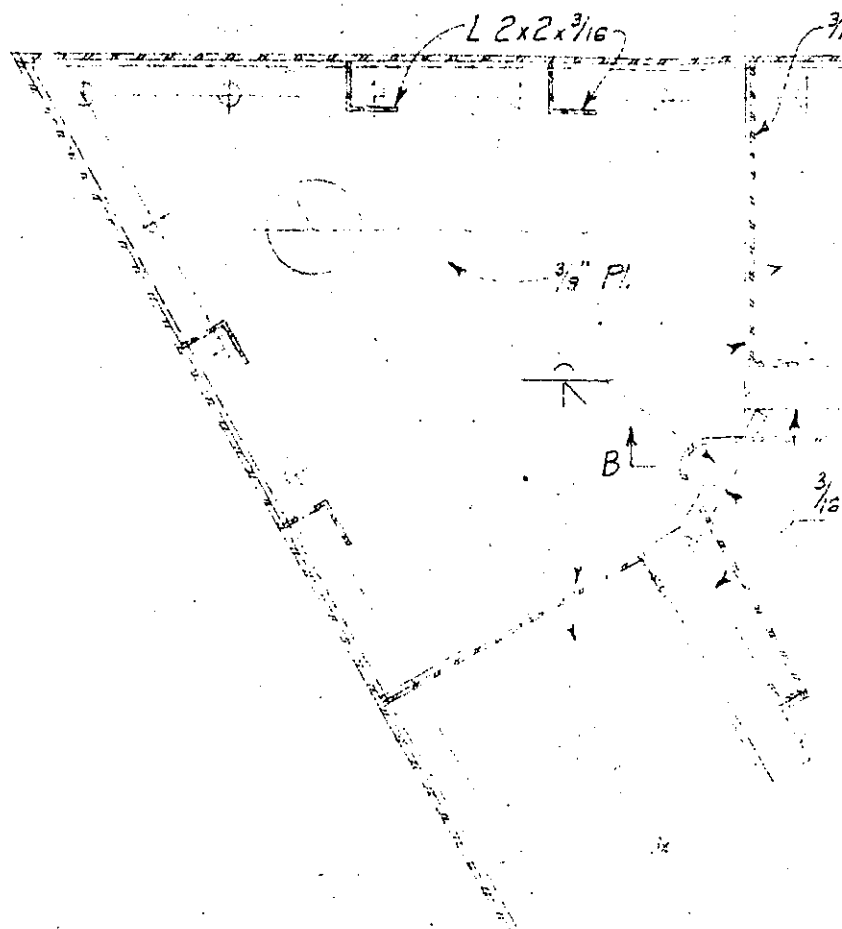
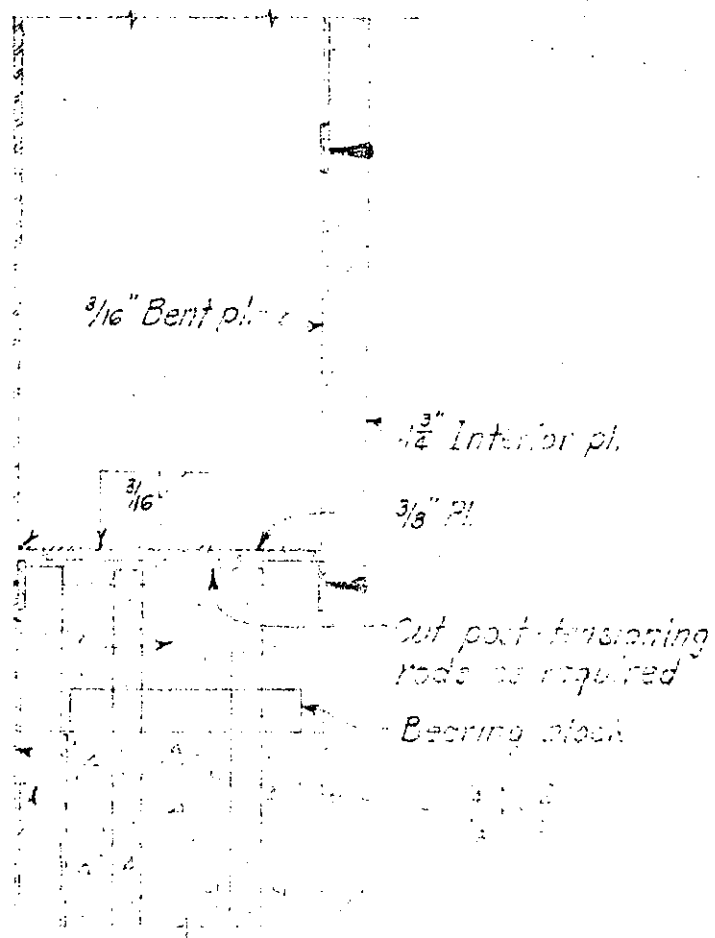
SECTIONAL ELEVATION
THRU WALL

SECTIONAL ELEVATION
NEAR CORNER



SECTION A-A

Note: For info, note
45 not shown in 50

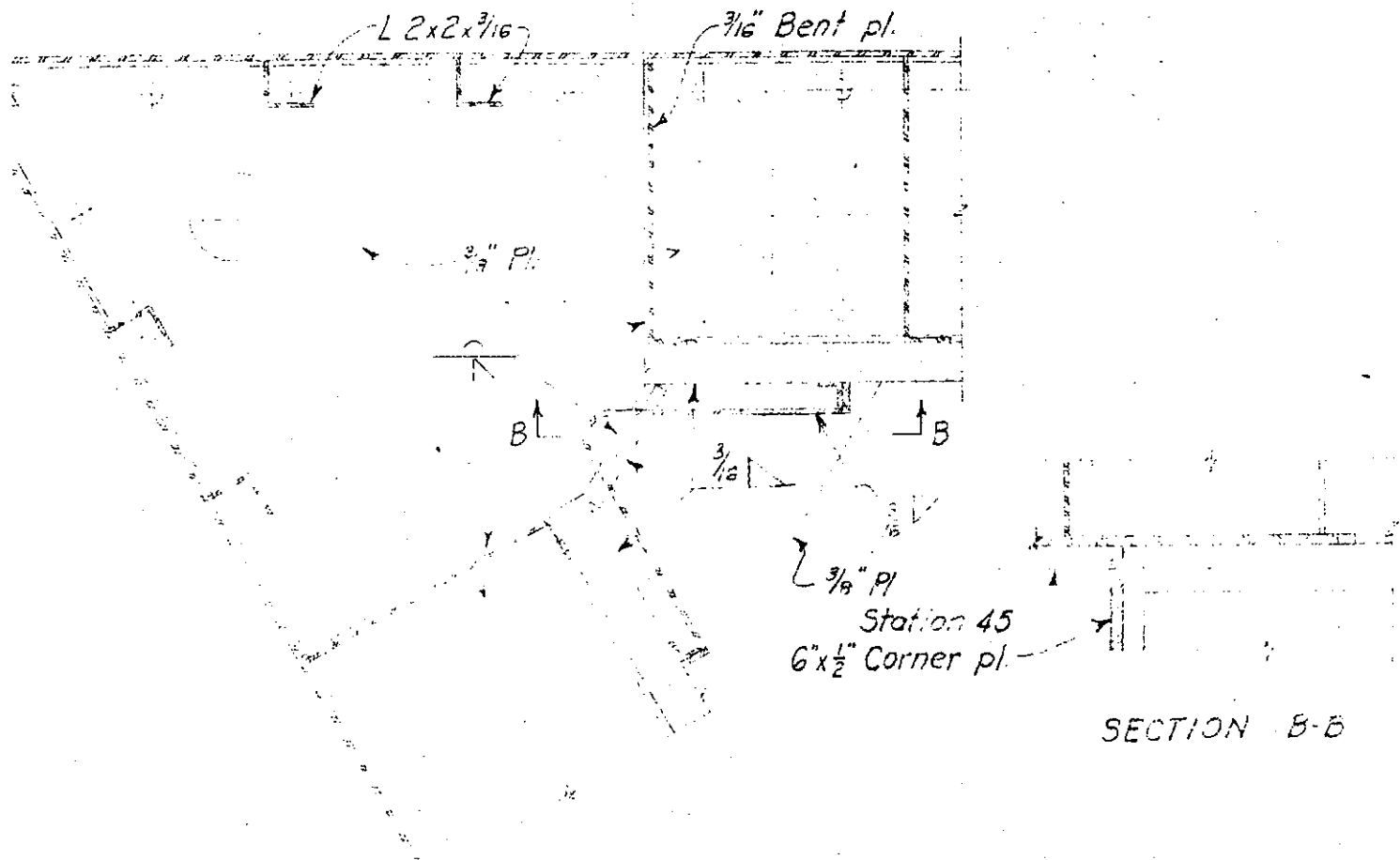


SECTIONAL PLAN AT C

SECTIONAL ELEVATION
NEAR CORNER

Note: For detail above Station
 45 not shown and Dwg X-CA-D-904.

5.
 10" ctrs.



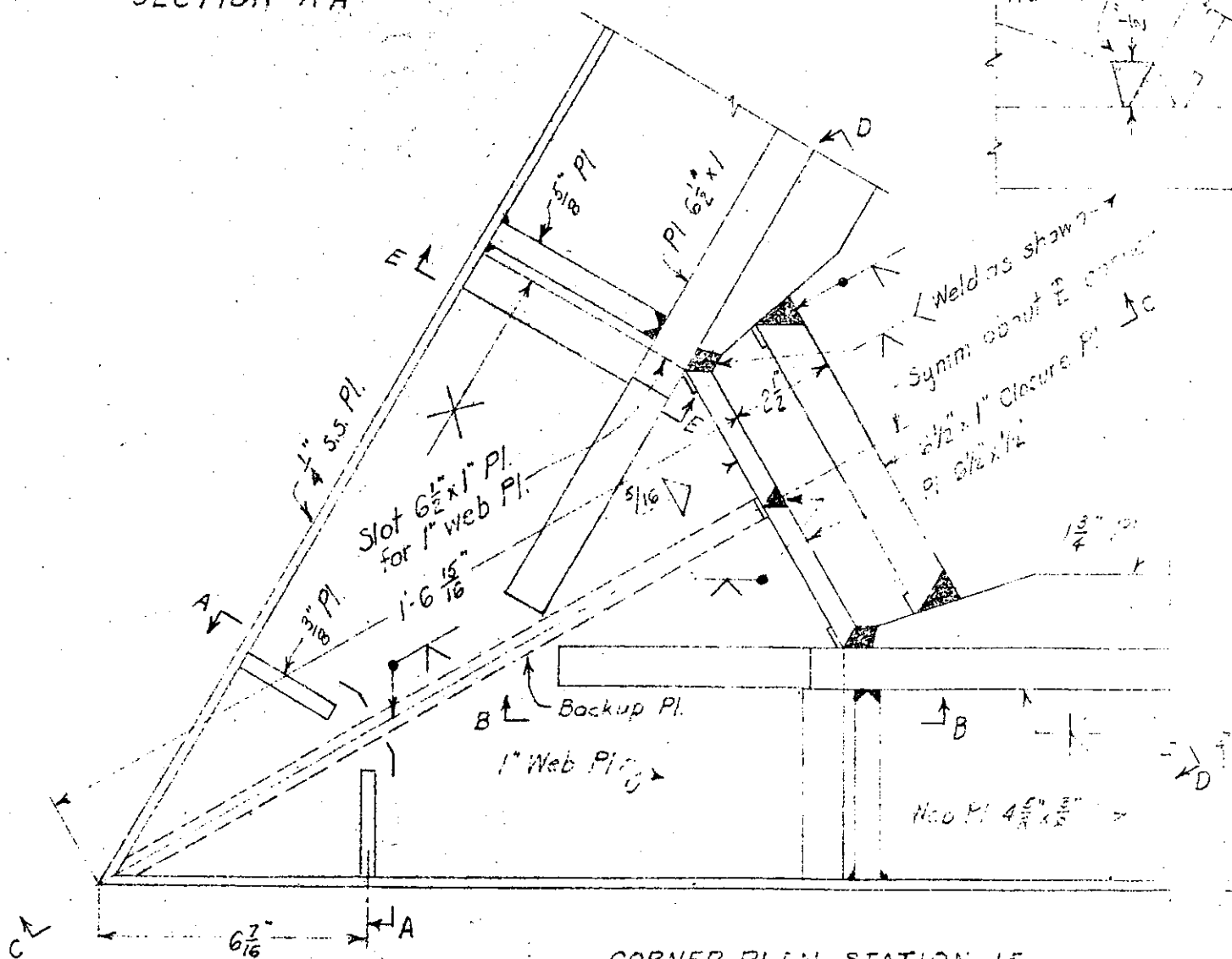
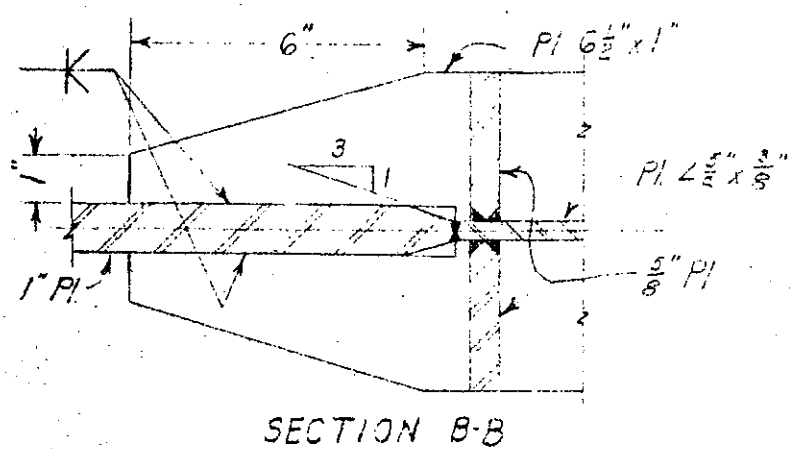
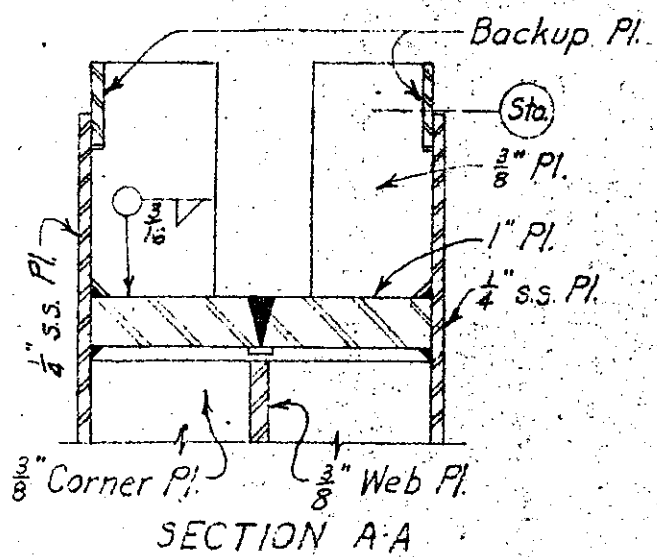
SECTION B-B

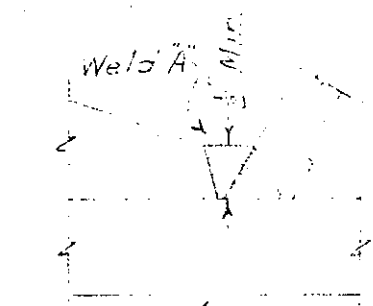
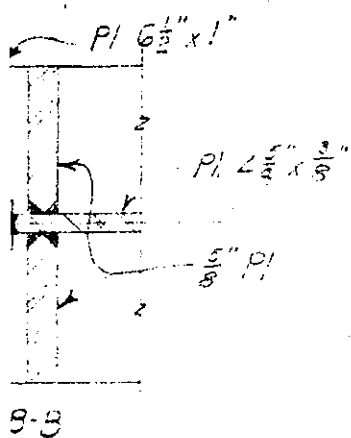
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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
JNEM GATEWAY ARCH
FIELD CONNECTION DETAILS
STATION 45

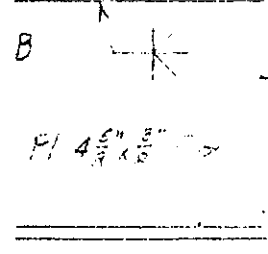
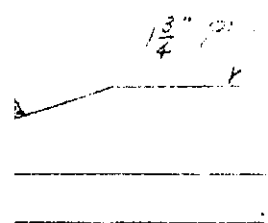
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CHECKED L.A. V. APPROVED

DENVER COLORADO AUG 3, 1964 X-O-A-2-906

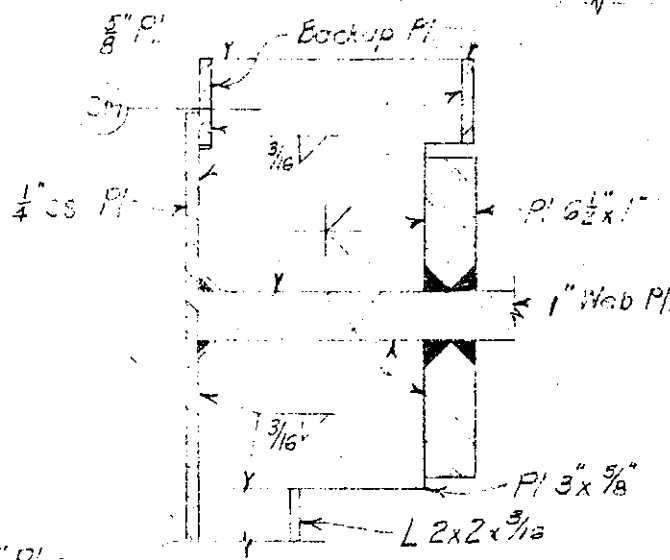
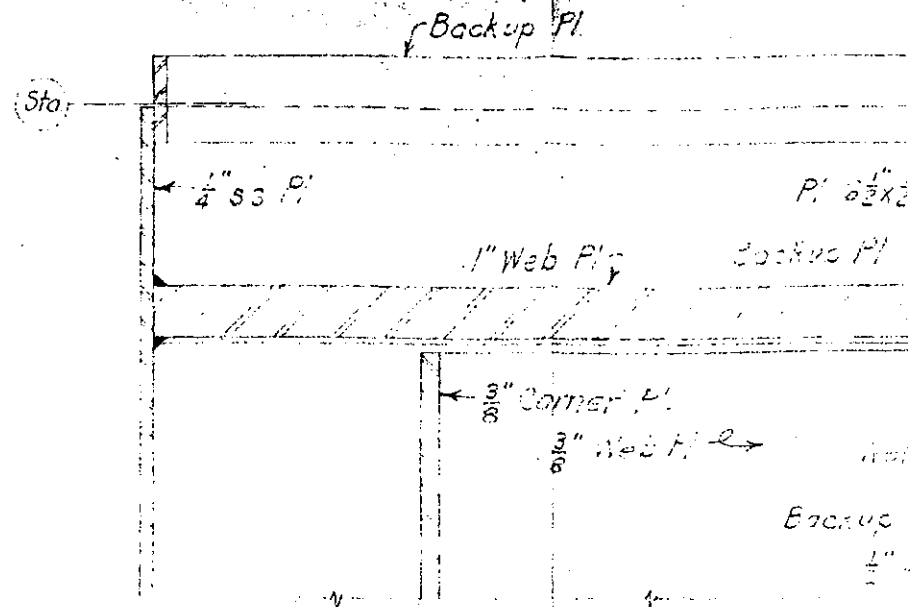




eld as shown -
 in about 1/2 inch
 1" Closure Pl
 6 1/2 x 1"



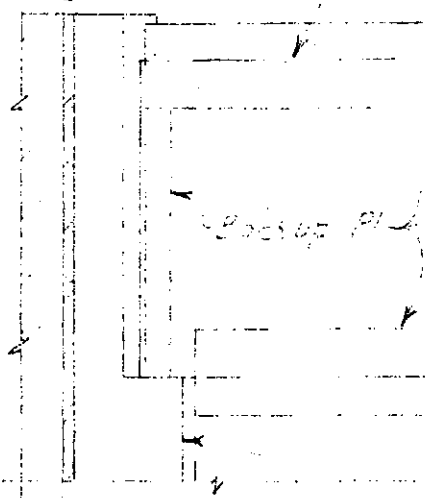
15



SECTION E-E

ST7W17

SECTION C-C



SECTIONAL V

11-2-64
D. CO.
J.N.
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DRAWN...
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on with
er.

Pl. stif. $5 \times \frac{3}{4}$

ST 15 WF 49.5

al. pl.

on with
rier.

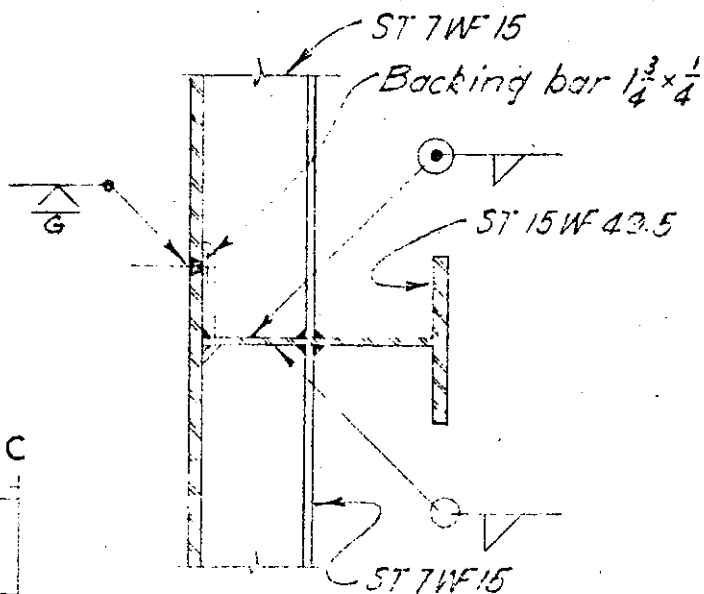
$5 \times \frac{3}{4}$

ST 15 WF 49.5

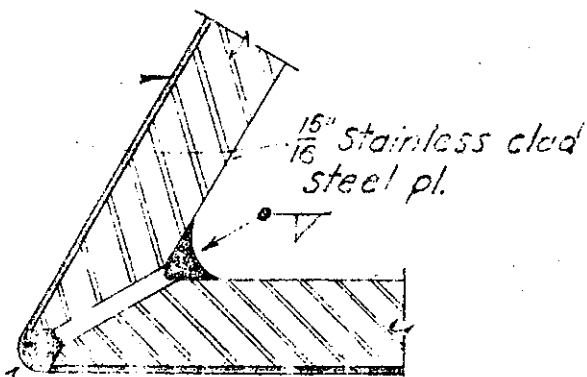
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D

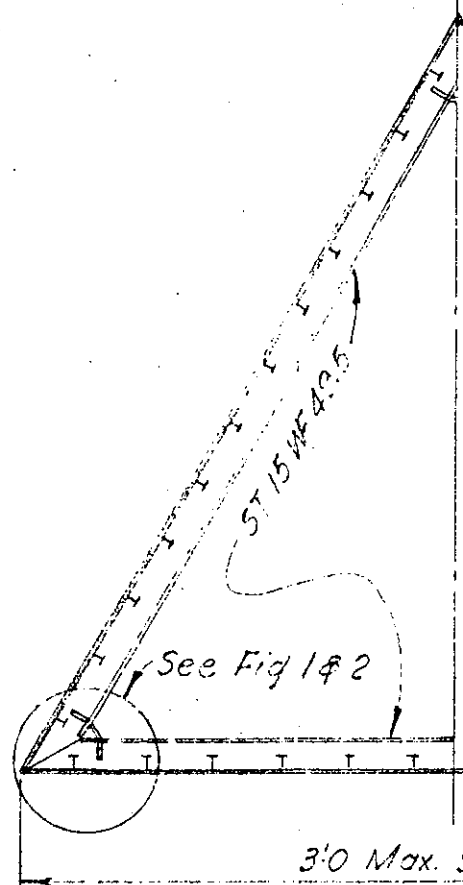
DESIGN)



SECTION C-C



DETAIL A



SECTION AT FRAME

PLAIN
INSULATION N

See Note A

#10 Ga. min.
stainless
steel skin pl.

#10 Ga. min. stainless
steel skin pl.

1/4" Stud bolt,
with 1" washer
in 9/16" hole.

15/16" Pl.

Field splice

Backing

Backing
bar 1 3/4 x 1/4

NOTE A Fill $1\frac{1}{2} \times \frac{5}{16} \times 0\frac{1}{2}$ (tack weld
in place). Fills may be of different
thickness to maintain flat smooth
skin pl.

DETAIL B

SECTION D-D

5

g bar $1\frac{3}{4} \times \frac{1}{4}$

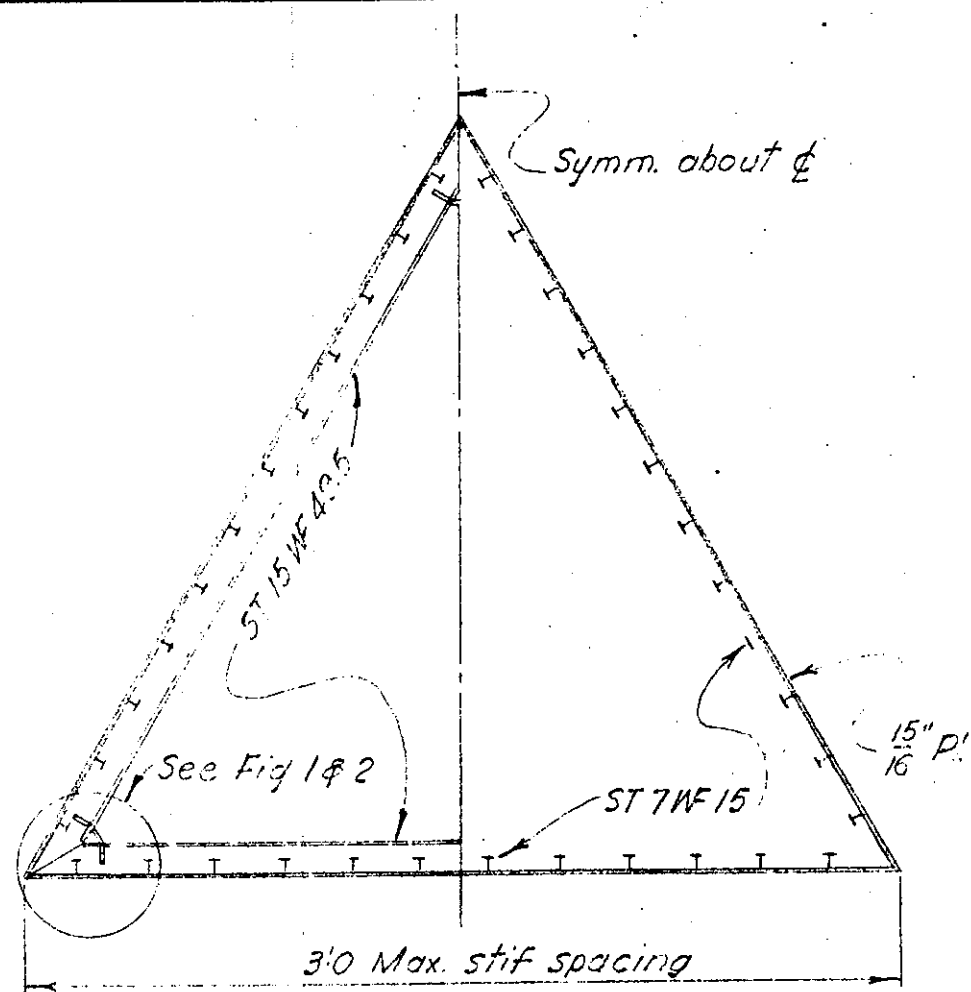
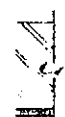
7

7' 15W 42.5

7

5

stainless clad
st pl.



SECTION AT FRAME

SECTION BETWEEN FRAMES

PLAN

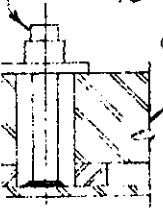
See Note A

INSULATION NOT SHOWN

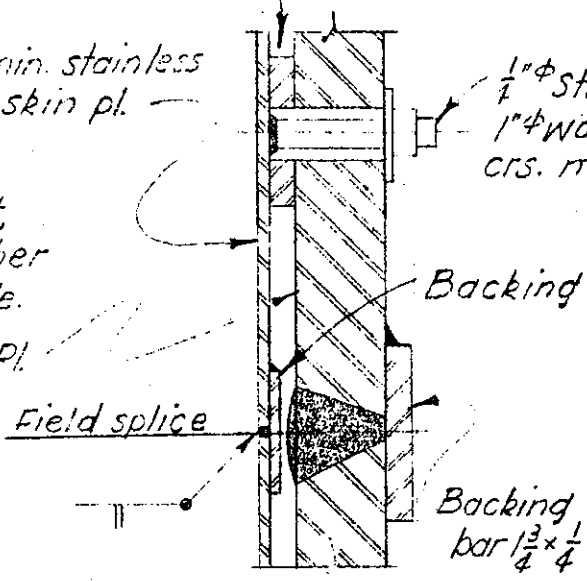
#10 Ga. min. stainless
steel skin pl.

$\frac{1}{4}$ " ϕ stud bolt with
1" ϕ washer @ 2'0
crs. max. $\frac{9}{16}$ " ϕ Hole.

$\frac{1}{4}$ " ϕ stud bolt,
with 1" ϕ washer
in $\frac{9}{16}$ " ϕ hole.



(track weld
e of different
flat smooth
AIL B.



SECTION D-D



ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

JNEM GATEWAY ARCH
PROPOSED REDESIGN
ARCH SECTION ABOVE STATION 45

DRAWN. JGA. SUBMITTED. H. Colander
TRACED. L.F.H. RECOMMENDED.
CHECKED. C.A.I. APPROVED.

DENVER, COLO. NOV. 23, 1964 X-OA-D-913

107-42-930

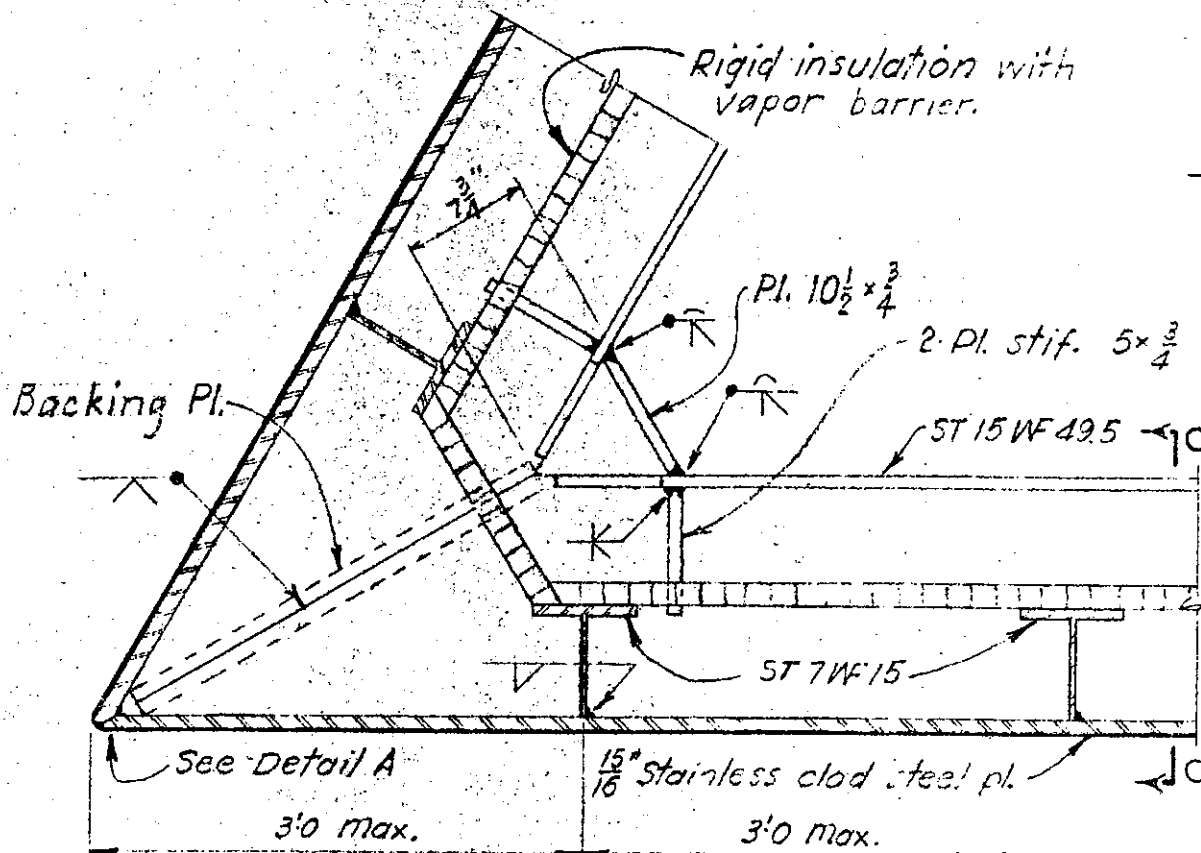


FIGURE 1
STAINLESS CLAD STEEL PL. DETAIL

NOTE: Details not shown same as Figure 1.

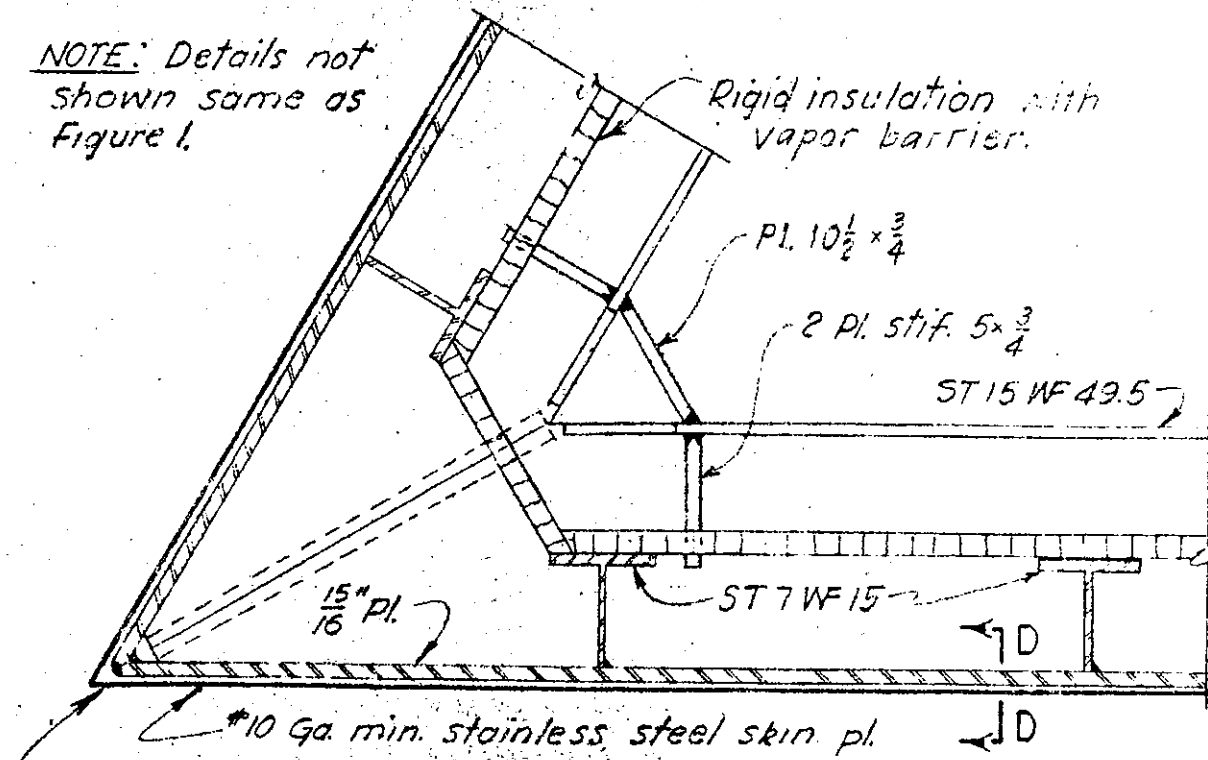
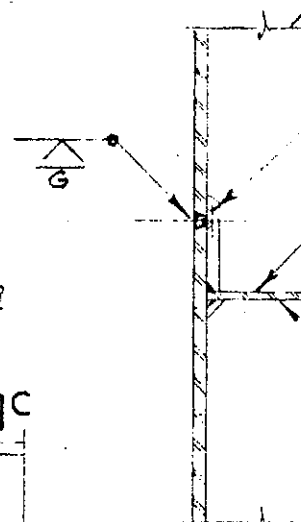
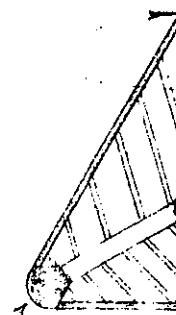


FIGURE 2 (ALTERNATE DESIGN)
STAINLESS STEEL SKIN PL. DETAIL



SECTION C



SECTION D

#10 Ga. min. stainless steel skin pl.

NOTE: Fill in place. F thickness to skin pl.

Chapter VI

ARCH ANALYSIS--CONSTRUCTION CONDITIONS

1. General

As indicated on the drawings of the proposed method of erection, reference 24/, the two legs of the arch are to be constructed simultaneously as cantilevers to a height of 528 feet (Station 21). A temporary horizontal bracing truss, 255 feet in length, is then to be inserted between the north and south legs at Station 22. After each end of the temporary bracing truss is rigidly attached to the adjacent arch leg by a connecting harness, construction of the arch legs will be continued to the crown, except the closure section. Next, the jacking frame and device will be installed. In this connection, we requested a detailed drawing but no details were received. To complete our studies, we have assumed that the following procedure will be used. Sufficient jacking force will be applied at the crown and held to allow disconnecting the temporary truss and lowering it to the ground. Then the jacking load will be increased to the required closure force, and this load held by temporary connections, while the closure section is welded in place. This required jacking load must be computed to allow for temperature variation and the effect of the creeper crane and crane supporting track loads or any other temporary load.

Stresses in the structure as presently designed were determined for the following stages of construction:

- (1) Each leg of arch erected to Station 45, post-tensioning completed
- (2) Each leg of arch erected to Station 21 as a cantilever, bracing truss being lifted into place
- (3) Each leg of arch erected to Station 1, bracing truss in place, creeper cranes at Station 11

The basic arch data and assumptions for these analyses are the same as those given in Chapter IV, supplemented by the following additional data:

Weight of bracing truss	87.6 kips	<u>24/</u>
Weight of connecting harness	49.0 kips	<u>24/</u>
Weight of creeper crane	186.0 kips	<u>25/</u>

Weight of crane supporting track

25/

Stations 9 to 19 5.60 kips

Stations 20 to 33. 6.53 kips

Stations 34 to 71. 7.35 kips

2. Loading Combinations

The following combinations of loading during construction were considered in the analysis. All temperatures are in degrees Fahrenheit.

Construction Case 1 Temperature Drop

Wind velocity at 30 feet. 50 mph

Temperature--exterior steel plate . . 20

Temperature--interior steel plate . . 20

Temperature--fabrication. 70

Construction Case 2 Temperature Rise

Wind velocity at 30 feet. 50 mph

Temperature--exterior steel plate

Extrados side (south leg) 125

Other sides 90

Temperature--interior steel plate

Extrados side (south leg) 100

Other sides 90

Temperature--fabrication. 40

3. Stress Analysis

Digital computer programs, developed for the analysis of the arch in its completed form, were used to perform all of the computations described below. Results were spot checked by desk calculator methods.

(a) Longitudinal Stresses Above Station 45

Until the bracing truss has been made an integral part of the structure, longitudinal stresses may be determined by a simple determinate analysis. However, determining stresses due to loads applied after the bracing truss has been connected requires an indeterminate solution. This was accomplished in the following manner:

(a-1) The three-dimensional stiffness of the bracing truss and its connecting harness were determined by the general purpose indeterminate analysis program. Rigidity of the joints in the truss was considered where connection details permitted.

(a-2) The stiffness of the bracing truss and the harness were combined with the stiffnesses of the arch legs, constructed to Station 1, to determine the forces induced in the truss due to the following loads:

1. Dead load due to weight of the arch above Station 21
2. Creeper load due to the creeper crane at Station 11 and its supporting track from Station 9 to Station 21
3. Wind from the west, acting on the entire arch and the truss
4. Wind from the south, acting on the arch legs only

(a-3) Once the forces in the bracing truss were known, longitudinal stresses were determined by simple statics.

(b) Combined Plate Stresses Above Station 45

The change in plate direction at each arch station will produce transverse stresses, as described in Chapter V. However, since they are small when compared with the transverse temperature stresses, they were not considered for the construction conditions. Combined plate stresses during construction were computed for the locations shown on Drawing No. X-OA-D-912, using the longitudinal stresses and the transverse temperature stresses. Minimum factors of safety against stress failure, described in Chapter V, were also computed.

(c) Longitudinal Stresses Below Station 45

The influence of post-tensioning, creep, and shrinkage is considerable in the lower portion of the arch. In order to determine the

stresses through the successive stages of construction, the superposition method of creep analysis was used. As previously stated in Chapter IV, accurate information on post-tensioning loads and sequence of tensioning, and the creep and shrinkage properties of the concrete are important to establish stresses due to these effects. Although we requested this information, it was not available 22/. We proceeded on the basis of the specific arch concrete information we did receive, and information from the Bureau laboratories.

The following creep function, fitted to a creep curve estimated by the laboratory, was used for the analysis:

$$\text{Creep} = 0.25 \left[1 - e^{-0.1(t-t_0)} \right] + 0.9e^{-0.21 t_0} \left[1 - e^{-0.2(t-t_0)} \right]$$

where t_0 is the concrete age (in months) when loaded

and t is the concrete age for the desired creep factor.

Shrinkage was assumed to vary linearly from 0 to 0.000,170 over a period of 12 months.

Since the age of the concrete when loaded is required in the above equation, the following construction schedule was estimated and assumed in the analysis:

Arch construction began May 1963
 Arch cantilevers constructed to Station 45. . . September 1964
 Arch cantilevers constructed to Station 21,
 bracing truss in place. February 1965
 Arch constructed to crown, closing force
 applied July 1965

More detailed discussion of this analysis is presented in Appendix F.

4. Results

The maximum longitudinal stress computed above Station 45 for the construction loads occurred at Station 27 and amounted to 17,000 psi

compression on the exterior plate at location NE1. From Drawing No. X-OA-D-912, the maximum longitudinal stress at Station 27 for normal loading conditions on the completed arch is 20,000 psi compression at EW1. The minimum factor of safety against stress failure during construction is 1.8 at NE1 on Station 27. It is noted that the comparable factor of safety in the completed arch is 0.8 for the extreme conditions and 1.2 for the normal conditions.

Longitudinal stresses in the steel and concrete below Station 45 during construction do not exceed those reported in Chapter IV for the arch in its completed form. Maximum stresses computed by superposition when the concrete is 10 years old agree within 15 percent of the final stresses computed by the effective modulus method using the creep factors described in Chapter IV.

Transverse stresses, although not determined in detail, are considerably less than those computed for the arch in its final form. For this reason buckling was not investigated.

Since stresses for construction conditions are from 50 to 85 percent less than those for the normal loading conditions in the completed arch, they are not presented in detail in this report.

5. Comments

It is noted that reference drawings 24/ and data 25/ carry June 1962 through April 1963 dates. We requested up-to-date data, but no additional information was received. Where differences in data appeared, we used the latest information. For example, the estimated weight of the truss given in the data book is considerably larger than that determined from the drawings. This difference raises a question whether the values in the data book 25/ are preliminary. However, we must assume that the weights of creeper cranes and track loads given in this reference are correct.

The adequacy of the bracing truss and its connecting harness was not investigated.

Chapter VII

FOUNDATION ANALYSES

1. General

Loads applied to the arch are transmitted to the underlying rock through the massive concrete foundation shown on Severud-Elstad-Krueger Associates Drawing No. S102 3/. The overall dimensions of the foundation for each arch leg are 75 feet long by 95 feet wide by 44 feet high. Foundation sides are stepped in 5 feet every 10.5 feet in elevation.

A 9-foot by 11-foot gallery for the passenger train runs through the foundation structure. In addition, there is a 160-cubic yard cavity in the lower portion of the foundation 26/.

2. Stability

Factors of safety against overturning and flotation, and foundation base pressures were computed for the arch for erection conditions and for the completed structure. The foundation structure was assumed to act as a rigid body on an elastic rock. Dead load was applied to a cantilever leg, erected to Station 0, for the construction case. (Note that this is a very extreme condition that is not likely to occur.) North, south, and west wind loads, with a velocity of 75 mph at 30 feet, and the dead load were applied to the completed arch. In addition, a water surface elevation of 421 feet 27/ is assumed for the maximum flood condition.

The weight of the foundation, base pressures, and factors of safety were computed by a digital computer program that had been developed for powerplant structures 28/. The following results were obtained:

Minimum F.S. overturning = 3.4--Dead load on cantilever

Minimum F.S. flotation = 5.9--Construction loads and completed arch compression

Maximum base pressure = 110 psi (8 tons per square foot)--Dead load and wind on completed arch

Minimum base pressure = 7 psi compression (0.5 ton per square foot)--Dead load and wind on completed arch

The value of 3.4 for the minimum factor of safety against overturning is for a very extreme condition that is not likely to occur. Stability of the foundation structure is therefore considered adequate. The

maximum base pressure of 8 tons per square foot is also indicated to be within allowable limits for the limestone foundation rock.

3. Stress Analysis--Deep Beam over Train Passageway

The portion of the foundation structure directly below the arch intrados and above the curved train passageway is actually a very short deep beam of irregular cross section; see Figure VII-1. This beam which is subject to a heavy concentration of load from the arch intrados is actually a three-dimensional stress problem. However, due to lack of time and the fact the foundation is constructed, we made a two-dimensional stress analysis as follows to obtain an approximation of stresses in this region.

A vertical slice, normal to the X axis of the arch, was analyzed for the distribution of stress in the foundation structure between the bottom of the arch intrados corner and the top of the train passageway. Maximum wind loads and the dead load due to post-tensioning, creep, and shrinkage were applied. The analysis was performed by a digital computer program for plane strain problems, using the finite element method of analysis 29/. Localized concrete tensile stresses of about 900 psi are indicated near the top surface of this deep beam above the edge of the passageway. This region of the foundation may crack since the construction drawings indicate only nominal reinforcement. The post-tensioning steel in the lower portion of this deep beam will probably not improve this undesirable stress condition; in fact, it may result in increased tensile stresses.

In view of this undesirable stress condition directly under the intrados of the arch, we caution that regular inspections be made in this area for evidence of structural cracking of the concrete.

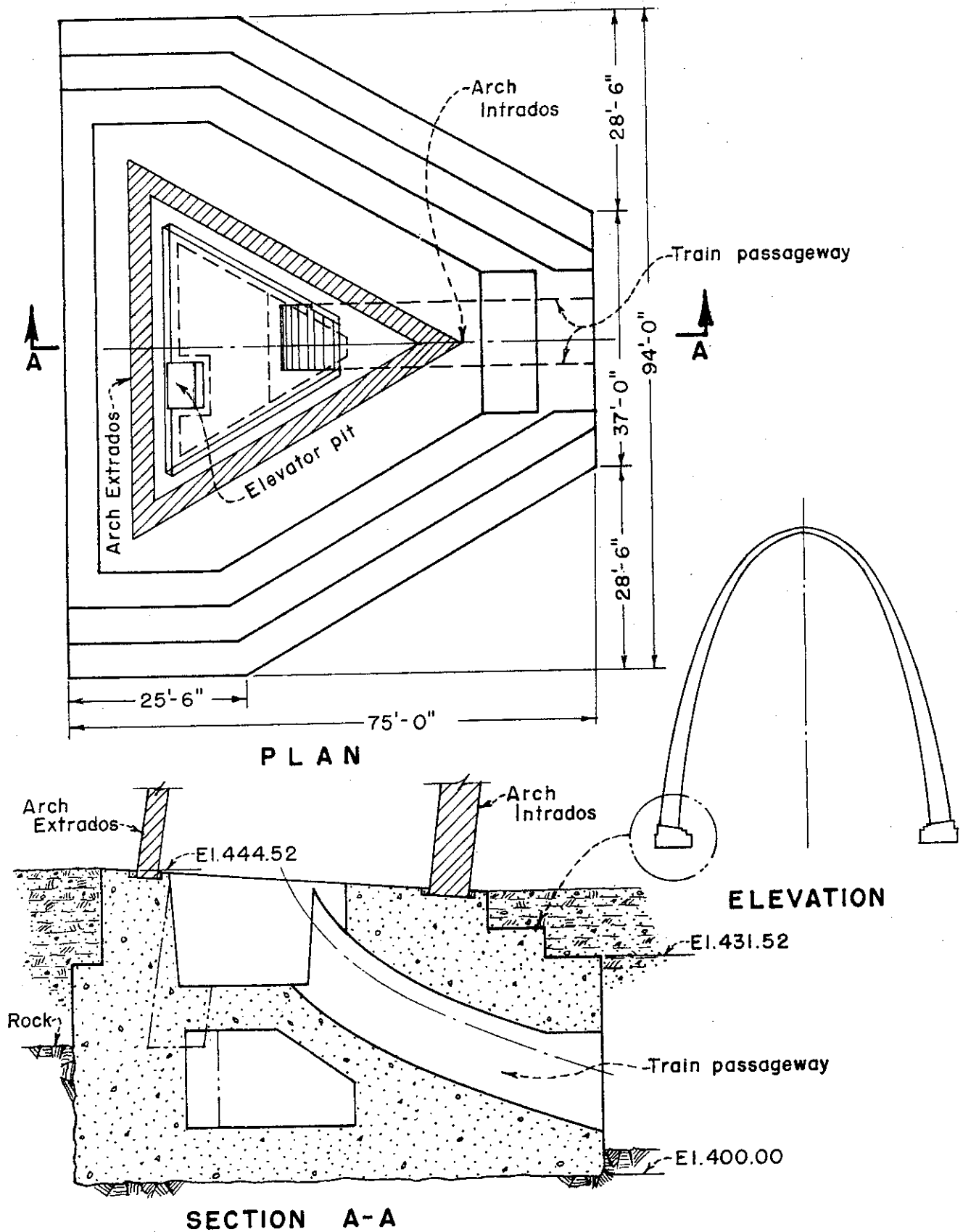


Figure VII-1
ARCH LEG FOUNDATION STRUCTURE

Chapter VIII

LIST OF REFERENCES

1. Severud-Elstad-Krueger Associates (SEKA) Data Book 3220
(Computations dated from September 1960 through February 1964)
2. Eero Saarinen and Associates Drawing No. AR-1 (revised
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3. SEKA Drawings No. S101-S-120 (latest revised drawings including
revisions dated April 4, 1964, on Drawings No. S113, S114,
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4. SEKA letter to Chief Engineer dated March 4, 1964 (memorandum
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21. Consulting Engineer Severud's letter of June 3, 1964, to Acting Chief Engineer, U.S. Bureau of Reclamation, discussing welding procedures
22. Consulting Engineer Severud's letter of May 5, 1964, to H. C. Olander, Office of Chief Engineer, discussing post tensioning data, spot welding tests, etc.
23. "Effective Flange Width of Stiffened Plating in Longitudinal Direction" by G. Murray Boyd, The Engineer, December 27, 1946
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27. Letter of May 19, 1964, from U.S. Weather Bureau, River Forecast Center, St. Louis, Missouri, to Chief Designing Engineer, U.S. Bureau of Reclamation
 28. "Stability Analysis," Electronic Computer Program No. SA-CPl2, by H. W. Anderson, Structural and Architectural Branch, U.S. Bureau of Reclamation, June 1963
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**U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
WASHINGTON, D.C. 20235**

**AERODYNAMIC STABILITY
OF
JEFFERSON NATIONAL EXPANSION MEMORIAL GATEWAY ARCH**

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and
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in collaboration with
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U. S. Department of the Interior**

July 1965

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AERODYNAMIC STABILITY OF JEFFERSON NATIONAL EXPANSION MEMORIAL GATEWAY ARCH

INTRODUCTION

The investigation herein reported describes wind tunnel tests on a full elastic model of the Jefferson National Expansion Memorial Gateway Arch to determine its aerodynamic stability in the wind.

This investigation was initiated at the request of the Bureau of Reclamation, U. S. Department of the Interior as an outgrowth of a review of the structural design of the Gateway Arch. This review included a study of the report of the late Dr. D. B. Steinman on the aerodynamic investigation of the original design, which indicated the need for some modifications and further aerodynamic tests to check the effectiveness of these modifications. As a result certain modifications were made, but there was no indication that their effect on the aerodynamic stability of the structure had been verified. It was decided that a structure of such importance, such unusual section, and such monumental proportions should be thoroughly investigated in order to assure its aerodynamic stability.

A preliminary conference indicated that the necessary investigation could be made in the special wind tunnel of the Bureau of Public Roads, and the work was undertaken under a memorandum of understanding between the Bureau of Reclamation and the Bureau of Public Roads, dated September 28, 1964.

BASIC THEORY FOR THE MODEL TESTS

This investigation involves the analytical determination of the wave forms and frequencies of a number of the lower modes of vibration of the structure in question, together with wind tunnel tests of a properly scaled dynamic model to determine the response of the structure to various wind conditions. In addition to the velocity range within which critical motion would be excited, the strength of the excitation can be determined, giving a direct indication of the rate of transfer of energy from the wind stream to the structure. This procedure was developed and has been used in the course of the investigation of suspension bridges over the past 20 years¹. The validity of the procedure for suspension bridges has been demonstrated a number of times by quite close correlation between the predictions from model tests and the actual behavior of the particular bridge.

It was recognized at the outset of this investigation that the Gateway Arch differed in several important respects from suspension bridges in both its structural and aerodynamic characteristics. Therefore a very careful re-examination was made of the basic laws of similitude on which these tests were conducted and interpreted.

Structurally, both the long span arch and the suspension bridge are non-linear in their response to load, and current methods of analysis recognize this problem. The non-linearity results essentially from the fact that the equations of equilibrium written for the undeformed structure are not valid for the deformed geometry under load. It can easily be shown that the bending moment in the stiffening truss of a suspension bridge may be expressed as:

$$M_i = M_{i0} - (H_L)y_i - (H_L + H_W)n_i$$

where

M_i = Moment at point i due to given live load
(The dead load moment is made zero by construction control)

M_{i0} = Moment at point i in an equivalent simple beam of same span length for given load

H_L = Cable tension due to live load

H_W = Cable tension due to dead load

y_i = Ordinate of cable at point i with respect to center of gravity of truss

n_i = Deflection of cable (and truss) at point i.

The last term is neglected in linear analysis, resulting in an overestimate of bending moment in the stiffening truss, which in some cases may be as high as thirty percent. When the corresponding equations are written for the arch, one obtains:

$$M_i = M_{i0} - (H_L)y_i + (H_L + H_W)n_i$$

In this case, the neglect of the non-linear term results in an underestimate of the bending moment in the arch rib.

In spite of this difficulty, the arch and the suspension bridge may both be considered as linear structures for purposes of dynamic analysis at the small amplitudes at which motion is initiated, provided that the effects of increasing amplitude are kept clearly in mind; namely, larger amplitudes tend to increase the stiffness in suspension bridges with consequent increase of natural frequency, while larger amplitudes in arches tend to reduce the effective stiffness and lower the natural frequencies.

It is also vital to remember that while the cable system is inherently stable for static loads, since an increase in deflection produces an increase of stiffness, the arch is subject to instability like a column, since increased deflections produce increased induced moment and a reduction of effective stiffness.

Aerodynamically, the particular arch under consideration differs from the suspension bridge in its relationship to the windstream in several respects:

- (1) The taper of the triangular shaped cross section will lead to a range of frequencies of vortex shedding instead of the common value across the full length of the span.
- (2) For certain wind directions the wind streaming past one leg can impinge on the other leg.
- (3) The curvature of the arch places the axis of any given short length at varying angles to the wind so that for a wind parallel to the arch axis, lower elements are nearly at right angles to the wind direction, but crown elements are parallel to the wind.
- (4) The most likely form of motion is perpendicular to the plane of the arch.

The first of these differences means that there is inherent in the model tests of the arch a tendency to produce a mixture of frequencies, subject to possible "capture" due to aeroelastic feedback. The second feature may produce strong buffeting effects on the downstream leg of the arch, so that the type of forces and frequencies in such action must be modeled. The third effect will modify air flow in both model and prototype in a similar fashion at the top of the arch where it is not expected that much in the way of a periodic dynamic force will be generated. The fourth effect indicates a form of motion independent of gravity forces.

The primary purpose of the model is to evaluate all these complex effects. General model theory demonstrates that the prototype behavior will be faithfully reproduced in the model if the laws of similitude are complied with; that is, the ratios of all classes of forces in the model are identical to the corresponding ratios in the prototype. The exact satisfaction of this condition is seldom attained, and if more than one force ratio is involved it is unlikely to be satisfied except at a unit scale of 1:1, that is, prototype size. Practical model work is accomplished by identifying the most important force ratio to be satisfied and designing the model to achieve this end. The other force systems are either corrected for or ignored. Ship model testing is a good example of the former, where model similitude is set to satisfy the ratio of gravity and inertia forces (Froude number), and corrections are computed for friction effects.

In establishment of the proper criteria of similitude it is thus necessary in the first place to have some insight into the nature of the forces which affect the behavior, although their exact patterns need not be known. The determination of these patterns and the forces which they produce is the fruit of the model test, in which connection the model has often been referred to as an analog computer.

Experience with suspension bridges and other aerodynamically instable systems has identified three phenomena as possible causes of such behavior: (1) Von Karman vortex shedding² (2) "flutter", a coupling of vertical and torsional modes,³ and (3) "negative slope" effects in which the lift-drag characteristics of the section are such that the resultant force may be in the direction of the motion transverse to the wind stream⁴. For all of these effects, the predominant forces are those due to pressure, inertia and elastic aspects of the compressible flow. The Reynolds number of the prototype flow for large structures in high wind velocities is in general on the order of 5×10^6 or larger, indicating that inertia forces are large compared to viscous forces. This would seem to indicate that proper modeling would be achieved if the other force ratios, pressure to inertia and pressure to elastic were satisfied. The first of these ratios is satisfied by modeling on the Froude number⁵, which for equal gravity fields in model and prototype requires:

$$\frac{V_m^2}{L_m} = \frac{V_p^2}{L_p}$$

or

$$\frac{V_m}{V_p} = \sqrt{\frac{L_m}{L_p}}$$

The ratio of pressure to elastic forces is dependent on the Mach number V/a where "a" is the velocity of sound in air. In the case under investigation, the velocities of both prototype and model are so small compared to the velocity of sound that this requirement may be safely ignored.

The assumption that the Reynolds number characterizing the viscous forces may be ignored must, however, be carefully re-examined. As is generally the case the similitude criteria above are derived on a basis of steady state flow. As such, they apply to periodic or transient effects only if the control of these effects does not involve other force systems. The flutter phenomena and "negative slope" effects can be described mathematically in terms of ideal non-viscous fluids and must, therefore, be independent of viscous forces. The Von Karman vortex shedding, however, cannot be so described, since the assumption of an ideal non-viscous fluid on a symmetrical section cannot produce anything but a steady symmetrical flow devoid of any lift force perpendicular to the wind stream.

The role of viscous forces (and possible surface tension effects) in vortex shedding is not quantitatively formulated. In any event, these forces play a secondary role in controlling not the magnitude but the frequency of the vortex formation in some cases. In this case they act in a role like a pilot valve in a servo mechanism, such that a small force controls and "triggers" a much larger force.

Research over the last several years has demonstrated the importance of this effect on air flows over large cylinders.^{6,7,8} The most recent work⁸ reports that the vortex shedding frequency is a direct function of velocity up to a Reynolds number of about 2×10^5 following the Strouhal criteria:

$$f = S \frac{V}{D}$$

with a value of S equal to approximately 0.20. Above this Reynolds number, vortex shedding was erratic. This is in general agreement with the earlier work of Roshko⁷, who also found that the region of uncertain and scattered frequencies extended to about 3.5×10^6 , after which vortices again became periodic at somewhat larger Strouhal numbers. Schmidt⁸ mentions this fact but does not elaborate. Schmidt's investigation of the details of behavior in the region of uncertain frequencies are very significant, since he not only obtained information as to the power spectrum of frequencies, but also the correlation measures of the action at various points along the axis of the cylinder, demonstrating the essential three dimensional aspects of the flows in this region of Reynolds numbers. He also demonstrated that the integrated lift coefficients associated with this region were an order of magnitude smaller than those at smaller Reynolds numbers. His work may be summarized as to the effects on circular cylinders as follows:

- (1) In the region of Reynolds numbers from about 2×10^5 to 5×10^5 there is a transition from sharply defined periodicity in the wake to an erratic and random shedding, with a corresponding drop of an order of magnitude of the effective fluctuating lift forces.
- (2) There is some indication of a return to periodicity at Reynolds numbers above 3×10^6 .
- (3) The critical Reynolds numbers and nature of the vortex frequency spectrum are both quite sensitive to minor surface imperfections.
- (4) Introduction of controlled disturbances by blowing air from holes in the cylinder at 30° points somewhat stabilized the action at high Reynolds numbers and again raised the lift coefficient to those typical of the subcritical range.

It is to be noted that Schmidt's model had a fundamental frequency of about 10 to 13 cps, whereas the periodic vortex frequency was most pronounced at about 30 cps; hence little aeroelastic feedback was observed. It is also well to quote two statements from this work. In paragraph 3.1, he states "These drag results, when combined with oil smear visualizations of the locus of flow separation points, confirm that the model was operated in a supercritical Reynolds number range", and in paragraph 3.3, "the effects of surface smoothness upon the rms values of lift load were found to be significant, since a consistent value (lower than the initial values) was obtained only after a considerable amount of polishing had been applied to the model."

All of the evidence, including that of the work of Schmidt discussed above, seems to indicate that the region of random frequency vortex action on circular cylinders is associated with the uncertainty as to the separation point of the flow. This point is sensitive to viscous effects on a smooth circular cylinder, as graphically demonstrated by tests of the entry of spheres into water by the Naval Ordnance Test Station in Pasadena, which show that for viscous flow the separation occurs about 15° ahead of the equator of the sphere and for artificially induced turbulent flow about 10° behind the equator. The uncertainty of separation point is, therefore, a boundary layer effect which is sensitive to both surface smoothness and the viscosity of the fluid, hence related to Reynolds number. The return to periodicity of vortex action would apparently correspond to the development of fully turbulent flow such that the separation point is changed to its rearward position but is no longer uncertain.

No such uncertainty exists with regard to sharp edged sections. Nevertheless, some observers have reported a decay in the strength and periodicity of the vortex formation at Reynolds numbers between 1×10^5 and 1×10^6 (See report of Professor Etkin⁹), while others have expressed opinions that it should occur, and still others have stated that their evidence indicates the contrary. Results obtained by Etkin on different sized models are themselves inconsistent when rectified to equivalent Reynolds number, a fact which he attributed to possible effects of the wind tunnel boundaries. Different aspect ratios of the two models may also be responsible (these ratios are not stated in his report), since it is quite likely that end effects are highly significant. Since the location of the separation point can not possibly be in doubt, some other effect sensitive to the boundary layer may be involved.

It is believed that the erratic movement of the position of the stagnation point may be involved, an assumption partially supported by the fact that the decay is much more rapid with increase of Reynolds number for a triangular section with blunt face forward. It is to be noted that the local flows in the vicinity of this stagnation point are hardly properly characterized by a Reynolds number based on the mean flow and the triangle edge, since they pass through a complete range of values from plus to minus, with a maximum value that associated with the circulation which must be present to produce the lift force. The rate at which the stagnation point can shift its position, with consequent change in the velocity in the boundary layer at a given point on the surface is a function of the rate at which energy exchanges can occur in this region. When the maximum velocities of circulation first reach the critical value for turbulent boundary development at the maximum point of a cycle, energy exchange rates suddenly become much larger. Since the local critical velocity is quite sensitive to surface conditions, it should be expected that it will be reached at rather random positions when the range of local circulation velocities is in the transitional range. When velocities become quite high, turbulent flow should be fully developed over nearly all of the cycle and the spatial distance over

which critical transition conditions^{exist} will be small. Such a conception is consistent with the observed data, which shows periodicity reappearing at high Reynolds numbers above 3×10^6 even for cylinders of circular section.

It is, therefore, believed that the model tests of this structure may best be done by modeling on Froude's Law, ignoring the Reynolds number, for the following reasons:

- (1) Previous research on suspension bridges, where an identical question exists with regard to girder bridges, has confirmed the validity of model tests by prototype observations.
- (2) Available data on even circular cylinders indicates that the range of erratic or random vortex action may be limited to the range of Reynolds numbers from about 3×10^5 to 3×10^6 . The computed Reynolds number for the Gateway Arch in a 60 mph wind at the 30' section is about 18×10^6 .
- (3) The tapered shape of the arch already produces a mixture of potential vortex frequencies in the model.
- (4) The "capture" of vortex frequencies by aeroelastic feedback is a very real possibility. It has been completely ignored in most of the experimental laboratory work showing random action, and could have little effect even in Schmidt's work, where the structural natural frequency was well below the predominant vortex frequency.
- (5) Where public safety is at stake, we prefer to be conservative.

The wind tunnel at the Bureau of Public Roads Fairbank Highway Research Station was designed for testing section models of suspension bridges. It is, however, quite suitable for testing a 1:120 scale model of the complete arch.

DESIGN AND CONSTRUCTION OF MODEL

To satisfy all the appropriate force ratios at a given geometrical scale ratio one may theoretically select a model material of appropriate properties and test in a fluid of any desired density. In a practical sense, however, one must work with available materials and with the fluid which is used in the wind tunnel; namely air at normal pressure and density.

This condition immediately sets the ratio for all force systems in terms of the inertia forces of the moving air stream. It follows that the scale of velocity is the square root of the linear scale, as is also the scale of time, while the scale of frequency is one over the square root of the linear scale.

The scale of unit stress, in pounds per unit area, is the same as the linear scale, and this applies to the modulus of elasticity, indicating the desirability of building the model of a material whose modulus of elasticity is $1/120$ that of steel. This is not feasible, but a ready solution is to scale the product EI at the fifth power of the linear scale for members subject to bending, and to scale the product EA at the cube of the linear scale for members subjected to direct stress. A material must be selected which will have a sufficiently low modulus of elasticity to permit the use of thicknesses sufficient to prevent local buckling. The material should also have sufficiently low damping to represent reasonably well the behavior of the prototype.

After considering several alternative methods of designing the model it was decided to make it of stressed skin design using Kodacel B2X, a cellulose acetate butyrate clear plastic sheet produced by Eastman Chemical Products, Inc. The manufacturer indicated that the modulus of elasticity ranges from 2.0 to 2.5×10^5 psi. Tests made on the material confirmed the lower figure which was used in designing the model. The stress strain diagram is essentially linear through a stress range sufficient for the purpose of these tests. Creep studies were conducted which showed that creep would not be a detrimental factor at the low stresses and relatively high frequency involved in the model arch. Following these tests a section model typical of the dimensions of the proposed model was fabricated and tested in bending and torsion to verify the computed elastic properties.

TESTS

The completed model was erected upon a substantial base mounted on a turntable to permit testing with the wind normal to the plane of the arch, parallel to the plane of the arch and at any intermediate angle. Movements of the arch at 14 locations were measured by means of clip gages made of electrical resistance strain gages cemented to beryllium copper springs. A closeup of one of these gages is shown in Figure 11-B. The clip gages were mounted on stiff frames downwind from the model arch. The free end of each gage was attached to the model by a nylon thread. The gages were shielded from the wind stream with cardboard containers to minimize vibrations of the gage itself. The strain gages were electrically connected through amplifiers to an eighteen channel oscillograph. Each clip gage was individually calibrated for displacement. Six of the clip gages were located in the plane of the arch axis measuring vertical movement (y axis) at stations 0 and 19 north and south and longitudinal movements (X axis) at stations 0, 19 and 45. Eight gages measured transverse motion (Z axis) at stations 0, 19, and 45 north and south and torsional motion at stations 0 and 19 north and south. Figures 7 and 8 show the instrumentation and test setup. (Stations correspond to those on prototype drawings by Severud-Elstad-Krueger Assoc.)

The measured frequencies of vibration and the values computed by the Bureau of Reclamation from the structural characteristics and mass of the arch prototype structure are shown in Table 1.

The first model tested and reported on in our interim report of April 1965 exhibited model frequencies about 25 percent below those computed for the first mode and about 10 percent below computed values for the other modes. This model is shown in Figures 9 and 10-B and the assembly drawing is shown in Figure 14.

It was assumed that the low frequency of the first model was due to local buckling of the individual panels and the corners in the critical sections of the arch from station 45 to about station 30. The model was rebuilt from station 45 to the crown with modifications as shown in Figures 7, 8, 10-A, and 15. This second model of the arch exhibited frequencies about 4 percent below those computed for all modes.

A correction was applied to the results of the tests for the discrepancy between the computed and observed model frequencies. This correction was based on a relationship which has been demonstrated repeatedly in the research in this field; namely, that the wind velocity necessary to cause oscillation of a given structure increases linearly with the frequency of the oscillation. The only factor significantly affected by this discrepancy in frequency is the predicted wind velocity of a given behavior. This can be corrected by multiplying the indicated velocity by the ratio of the computed to the actual model frequency.

In the still air tests the model was set in vibration in different modes by plucking it and recording the motion on the oscillograph. This indicated directly the frequency of the oscillation in a given mode and the amplitude at various points along the arch, and provided a record which could be analyzed to determine the structural damping of the completed model. The displacement form of those modes which were excited on the model corresponded quite closely to the computed forms of the respective modes.

In the still air tests of the first model the damping was measured and the average value of the logarithmic decrement was found to be about 0.18. The still air tests of the second model, with reduced local buckling, found the logarithmic decrement to average 0.155. These logarithmic decrement values were measured at a double amplitude of movement corresponding to 0.3 feet along the "Z" axis at the crown of the prototype. The logarithmic decrement ($\delta = \log_e \frac{X_{n-1}}{X_n}$ where X_{n-1} , X_n are amplitudes of any two successive cycles) may be interpreted as the rate at which oscillations decay due to energy dissipation.

The structural damping for small suspension bridges has been measured in the field and found to range from about 0.10 to 0.15 with a few examples in certain modes extending to higher levels. For long span suspension bridges it is probably of the order of 0.05. Measurements of the structural damping of simple span highway bridges give values of the order of 0.07 to 0.15. Theoretical analysis would indicate that the damping in simple spans arises largely in the bearings supporting the structure. Only a qualitative statement about the damping of this arch may be made. Its welded construction and the absence of any sliding support bearings indicate a probable low value of structural damping. On the other hand, the combined action of concrete and steel in the portion filled by prestressed concrete could well impose a relatively high level of damping. Some indication of the probable damping of the structure should be obtained from field tests of the completed structure, since this factor is quite critical in the computations which extrapolate model data to prototype behavior if the damping of both model and prototype are not the same.

The predictions of prototype behavior from the aerodynamic study also depend upon the precision of the computed frequencies of oscillation of the structure based upon its structural properties. In this case there may be some uncertainty concerning the effective interaction of the steel shell and the prestressed concrete, dependent upon the degree of prestressing and the effectiveness of the bonding or anchorage of the concrete to the steel.

RESULTS OF TESTS

On the basis of existing evidence it is believed that the excitation of the arch is produced by the well-known phenomenon of the von Karman vortex street. The frequency of vortex-shedding depends upon windspeed, the cross-sectional shape and, as indicated above, partly on the Reynolds number¹⁰. It is defined by the non-dimensional Strouhal number $S = \frac{fD}{V}$, where f is the frequency of shedding of vortex pairs, D is a width transverse to the wind stream and V is the wind velocity. For cross-sections with sharp edges the positions of flow separation are fixed by the edges and S is relatively independent of Reynolds number. When the vortex-shedding frequency is nearly the same as the natural frequency of oscillation of the structure, serious motion can result.

In Figures 1 through 6 are plotted the double amplitude of observed oscillation against wind velocity for different modes of oscillation and for wind angles from 0° to 90°. Both coordinates are plotted to prototype scale. The wind velocities have been corrected by 4 percent for the discrepancy in vibration frequency of the second model for all modes as mentioned above.

The modes are numbered and identified in Table 1. Only the first three modes are readily identified in the wind-induced motion of the model. The first mode is a lateral oscillation of the arch as a whole at right angles to the plane of the arch acting as a cantilever from the base. The second mode is a longitudinal horizontal motion of the structure in its own plane. The third mode is a twisting about the vertical axis through the crown of the arch. Each figure shows the response in the modes which developed in a wind from the direction indicated at the top of the figure. Each mode was measured at its point of maximum or near maximum amplitude, the first and second at the crown and the third at station 19.

A comparison of these figures shows that the principal excitation is in a wind parallel or nearly parallel to the arch and that the oscillations in wind normal to the arch is much less. As is to be expected the mode having the lowest natural frequency is the first to appear when the wind attains a velocity such that the vortex action about a dominating portion of the rib can act at or near resonance with its frequency. With increasing velocity this motion increases, but meanwhile the vortex frequency on parts of the arch approaches that of the third mode which makes its appearance at a wind velocity about 60 mph higher than that at which the first mode appears in a parallel wind.

The type of motion that both models exhibited in all modes was quite erratic, that is, movement would build and subside in irregular beats with neither the rate of increase or the maximum double amplitude of movement reached being predictable.

As the wind velocity was increased and the third mode became more prominent the two modes were always antagonistic as to natural frequency and vortex excitation, tending to mutual interference and inhibition. Nevertheless, it is seen that with further increase in wind velocity the amplitudes of both increased, indicating a net input of energy from the wind.

The oscillograph records shown in Figure 12 indicate the depth of this interference in the form of frequent phase shifting and amplitude changes as energy shifts from one mode to the other. The plotted curves show the net effect of the augmenting and interfering excitation of these modes caused by the taper of the ribs and the varying spacing of arch legs (controlling the timing and placement of buffeting vortices on the leeward leg) and influenced by the varying slope of the rib and thus the effective angle of attack of the wind.

The greatest double amplitude of movement in the first mode for the second model was about 30 inches at the crown at a prototype wind velocity of 90 mph. The first mode motion started at about 40 mph and built to a double amplitude of 6 inches at 60 mph. The third mode motion started at about 100 mph with the double amplitude increasing as the wind velocity was increased. The maximum wind velocity used during the tests represented 150

mph on the prototype. No attempt was made to test at a higher wind velocity because of the possibility of damaging the model.

The first model tested, which was the basis for the interim report, showed a maximum double amplitude of 36 inches at the crown in the first mode for a parallel wind of 86 mph. The prototype response curves for the first model are shown in Appendix A.

The double amplitude values plotted in figures 1 to 6 were obtained by noting the maximum movement in a particular mode of a 60 second test run at a constant wind velocity. The first model first mode double amplitude of 36 inches was reached only once in 60 seconds of model time, which is about 11 minutes of prototype time, whereas 75% of this amplitude or 27 inches was reached or exceeded 7% of the time and 50% or 18 inches was reached or exceeded 18% of the time. The second model first mode double amplitude of 30 inches was again reached only once in 60 seconds of model time but 75% of this amplitude or 22.5 inches was reached or exceeded 16% of the time and 50% or 15 inches was reached or exceeded 55% of the time. It is quite probable that if data were recorded for several minutes at any constant wind velocity that the double amplitude of movement could be slightly greater, but the percent of time that the greater movements would be maintained would become very small.

CONCLUSIONS

As a result of the model tests and evaluation of the limited field tests, it is concluded that:

1. Objectionable and potentially dangerous aerodynamic oscillations of the prototype arch for both the completed arch and the critical erection stages just before closure are highly probable under an unfavorable wind direction and velocity (it may be 10 or 15 years before such conditions occur, as was the case for the Golden Gate Bridge ¹¹).
2. The critical wind direction is parallel to the arch (north-south), and significant oscillations in the fundamental mode may be expected in wind velocities above about fifty miles per hour for sustained periods.
3. The energy levels involved in these oscillations are within the range where they can probably be successfully controlled by the introduction of additional damping.
4. Strain levels of fluctuating strain are not high enough to induce large damping absorption in the existing structural system.
5. The validity of the assumption that the model tests are overly conservative in that the exciting forces will be greatly reduced at high Reynolds numbers of prototype behavior is not supported with the conclusive evidence required where questions regarding public safety are concerned. This matter was fully discussed in the second section of this report.

RECOMMENDATIONS

It is recommended that:

1. A program of observations on the completed structure be established to watch for any indication of aerodynamic oscillation.
2. A system of artificial damping devices be designed so that they may be fabricated and installed at the first signs of difficulty.

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ACKNOWLEDGEMENTS

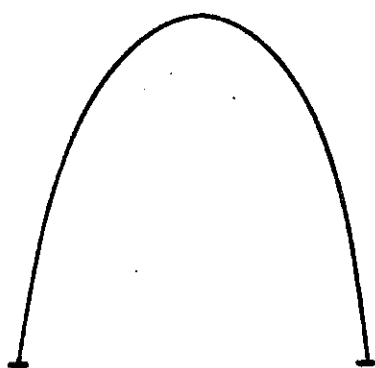
Appreciation is expressed by the authors to Mr. Donald L. Michael who assisted in the preparation of material for the final report, to Mr. Harry Laatz who assisted in setting up the instrumentation and conducting tests, to Mr. Paul J. Rees who did the layout, shop and detail drawings for the models, to Mr. Robert K. Taylor who machined and assembled the two elastic models and to Mr. George M. Davis who built a wooden model and the test platform for the aerodynamic tests.

TABLE 1

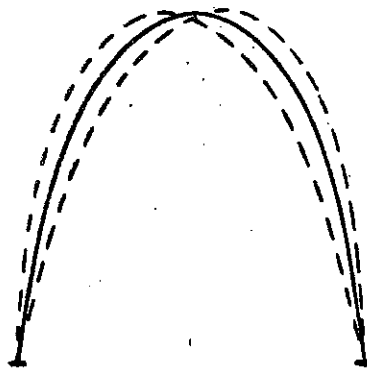
ARCH FREQUENCIES

Mode	Description	Frequency			
		Computed		Observed	
		Prototype	Model Scale	First Model	Sec. Model
1	Fundamental, normal to arch plane (Z axis)	0.485	5.31	4.2	5.1
2	Fundamental, in arch plane (XY plane)	0.737	8.07	7.4	7.8
3	Rotation about vertical axis thru crown	0.832	9.11	8.3	8.8
4	Second Mode in arch plane (XY plane)	1.219	13.35	*	*
5	Third mode normal to arch plane (Z axis)	1.300	14.24	*	*
6	Third mode in arch plane (XY axis)	1.785	19.55	*	*

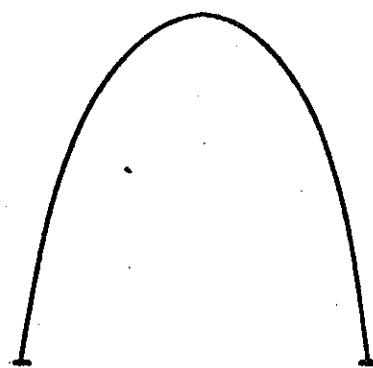
*Were not observed on model



First Mode



Second Mode



Third Mode

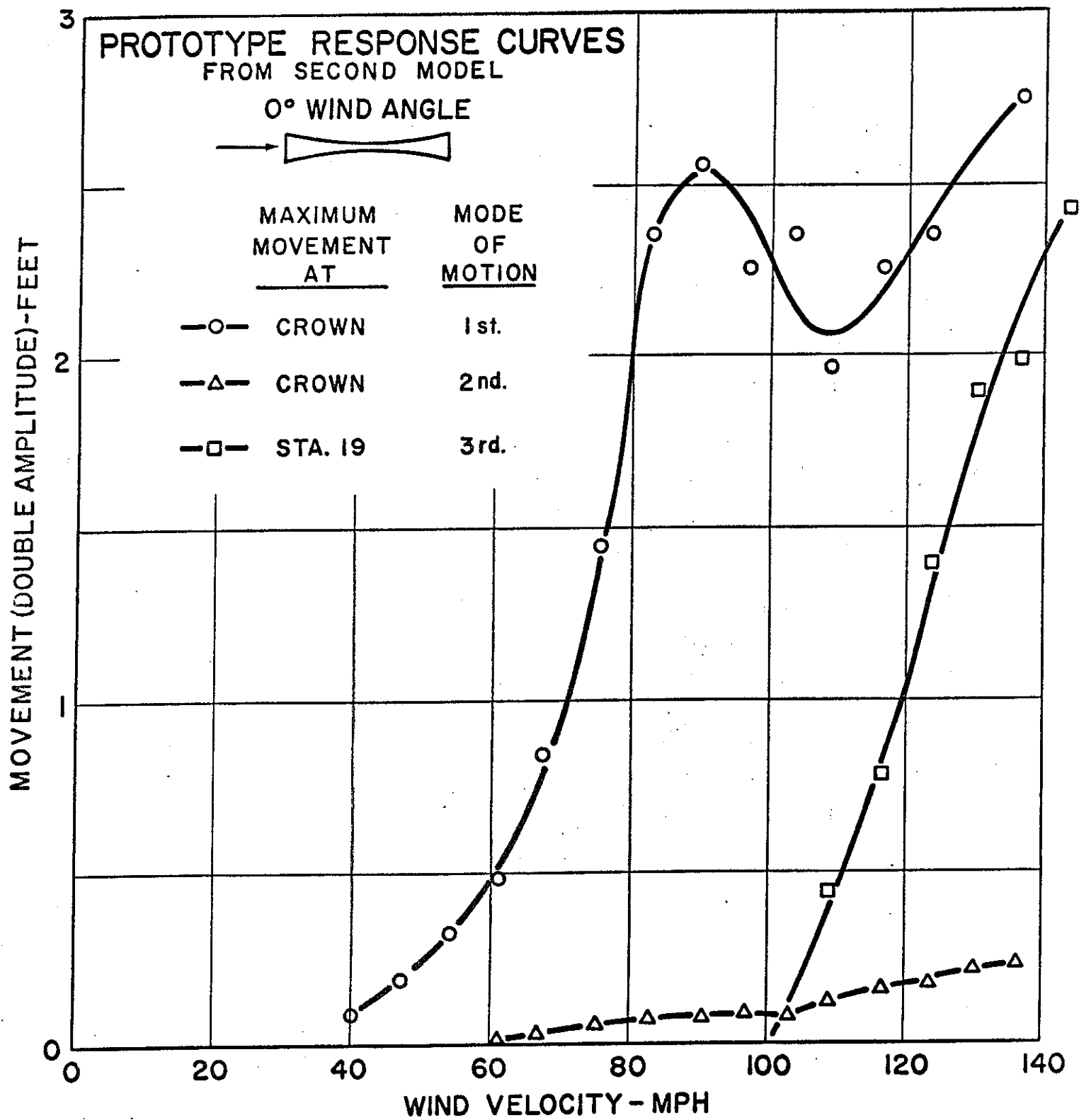


Fig. - 1

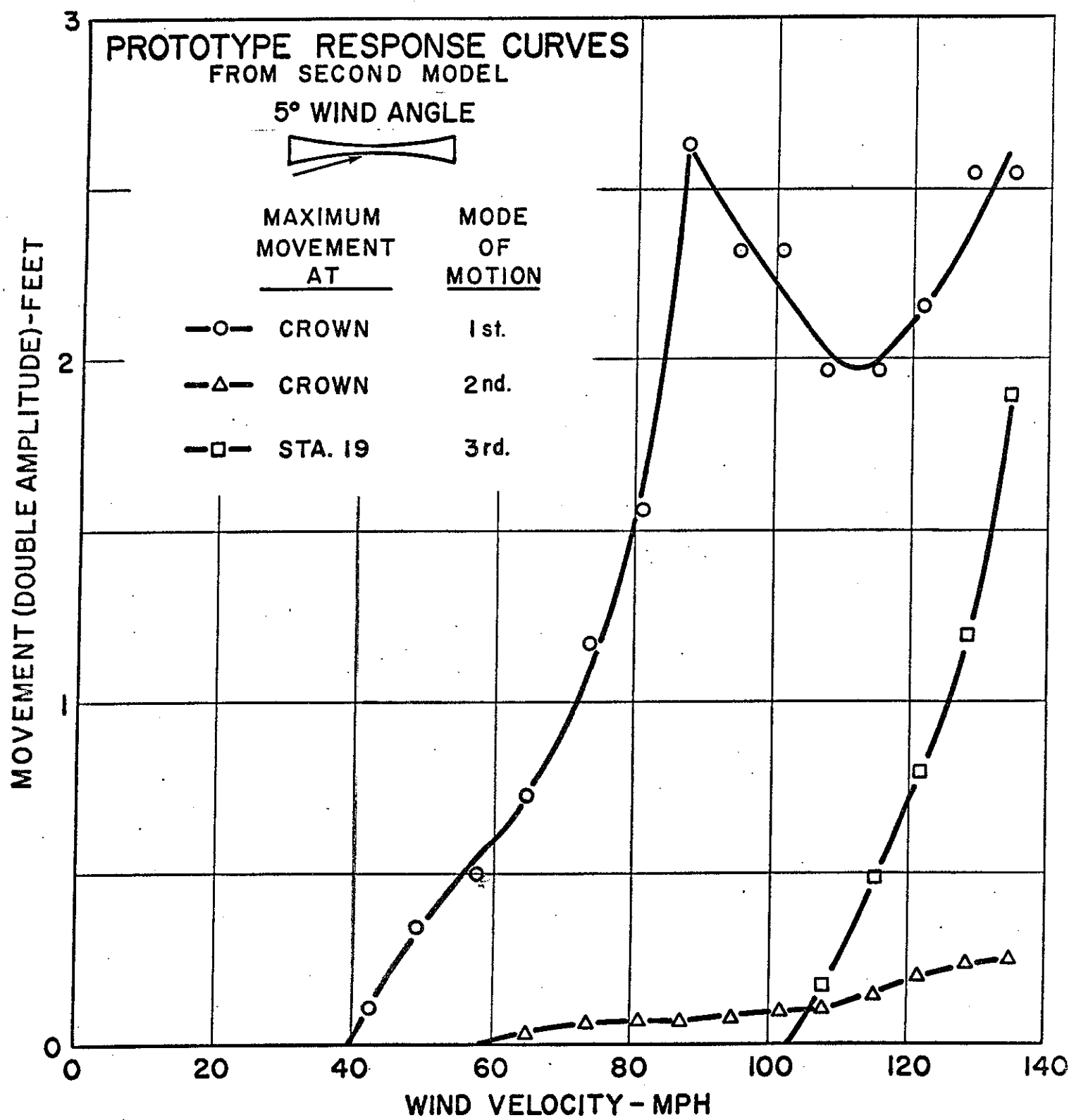


Fig. - 2

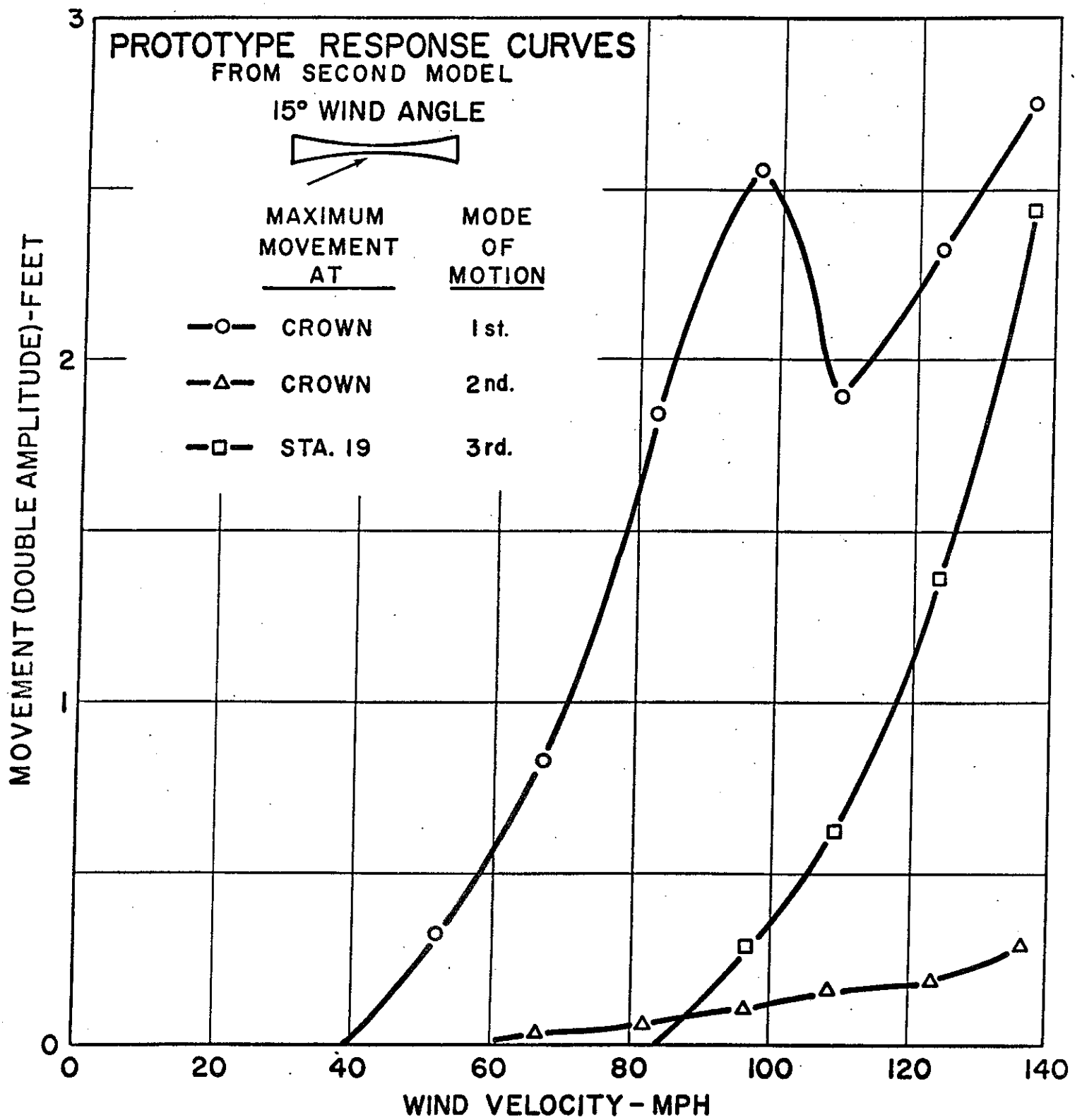


Fig. - 3

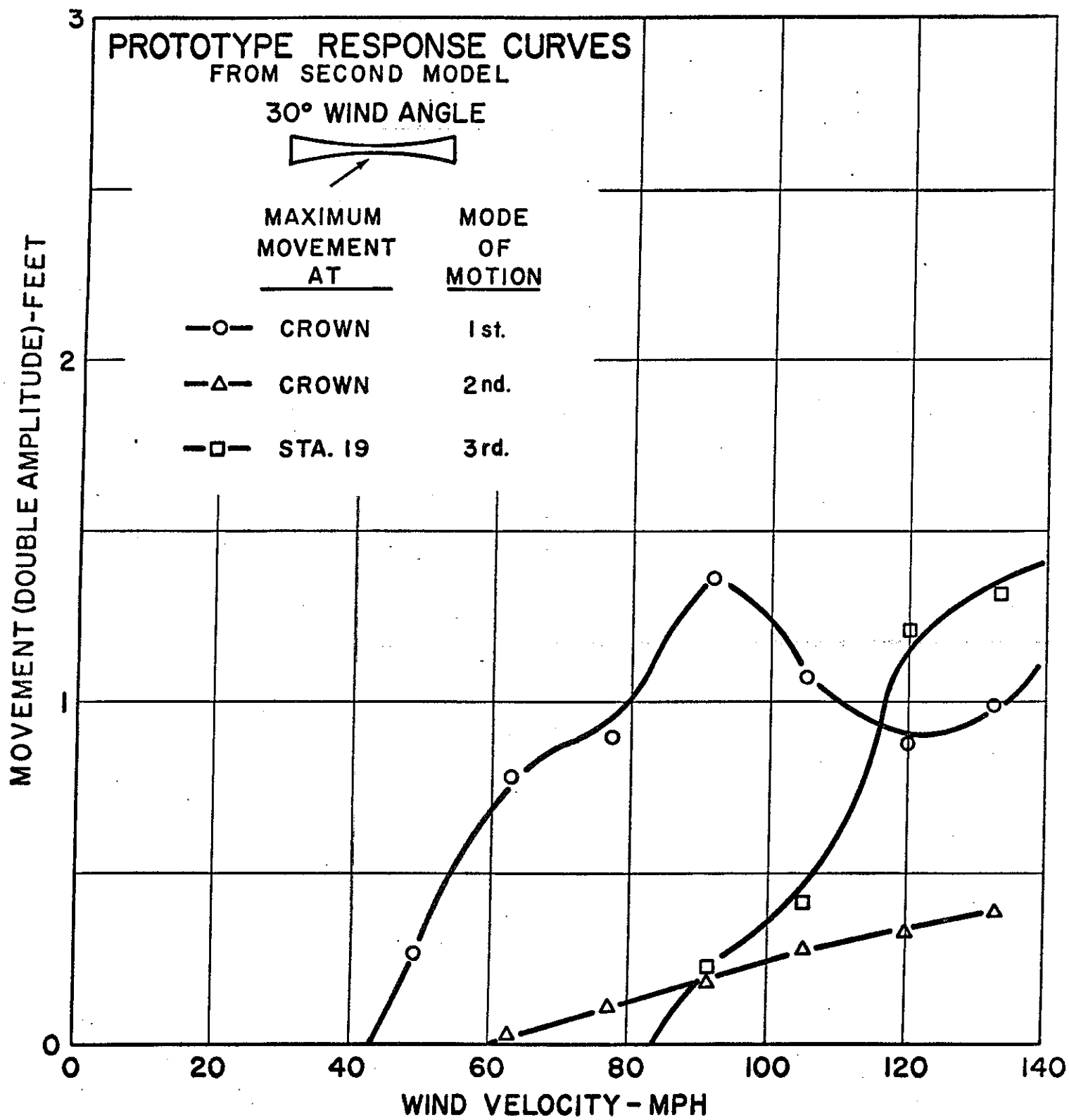


Fig. - 4

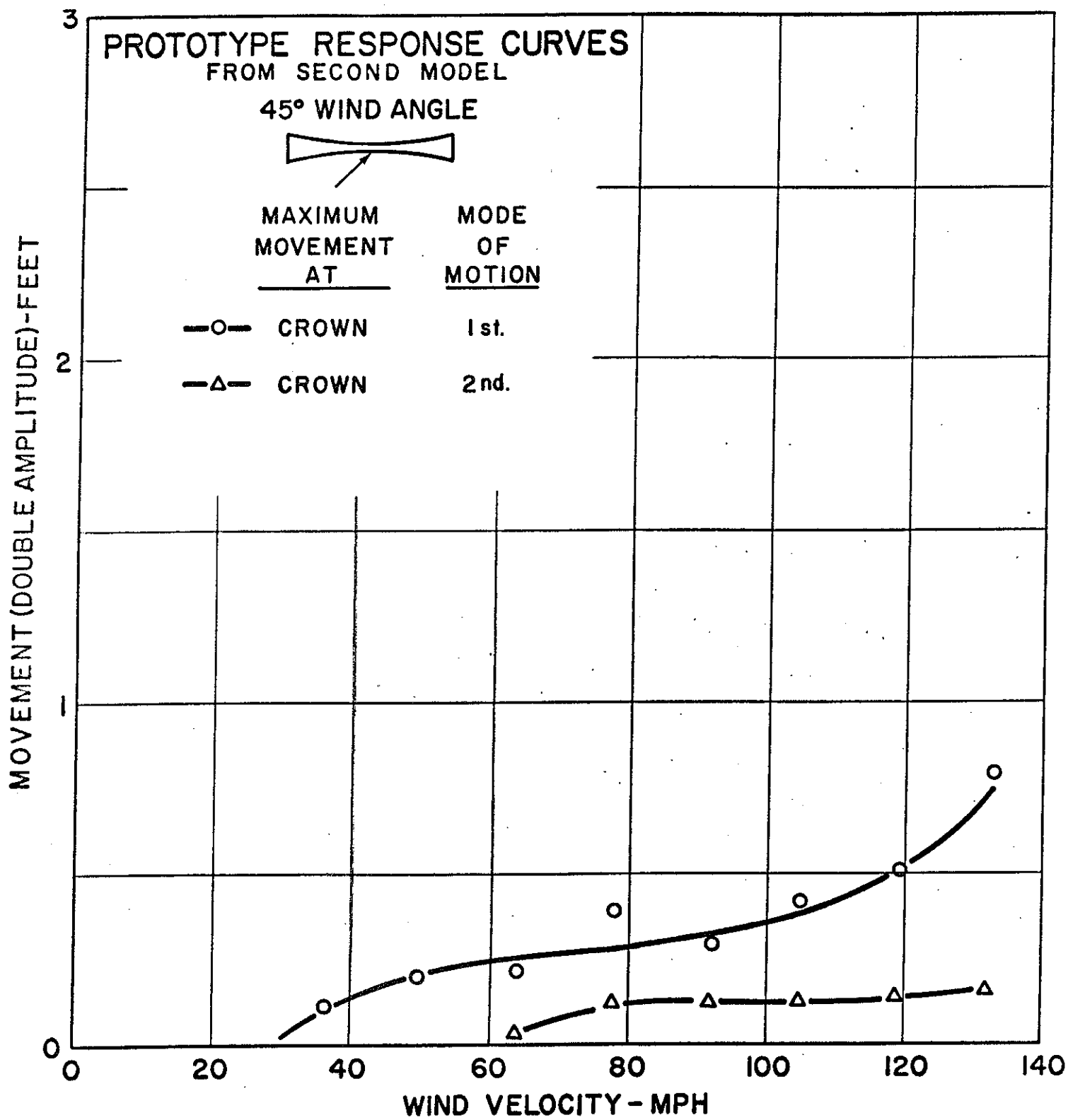


Fig. - 5

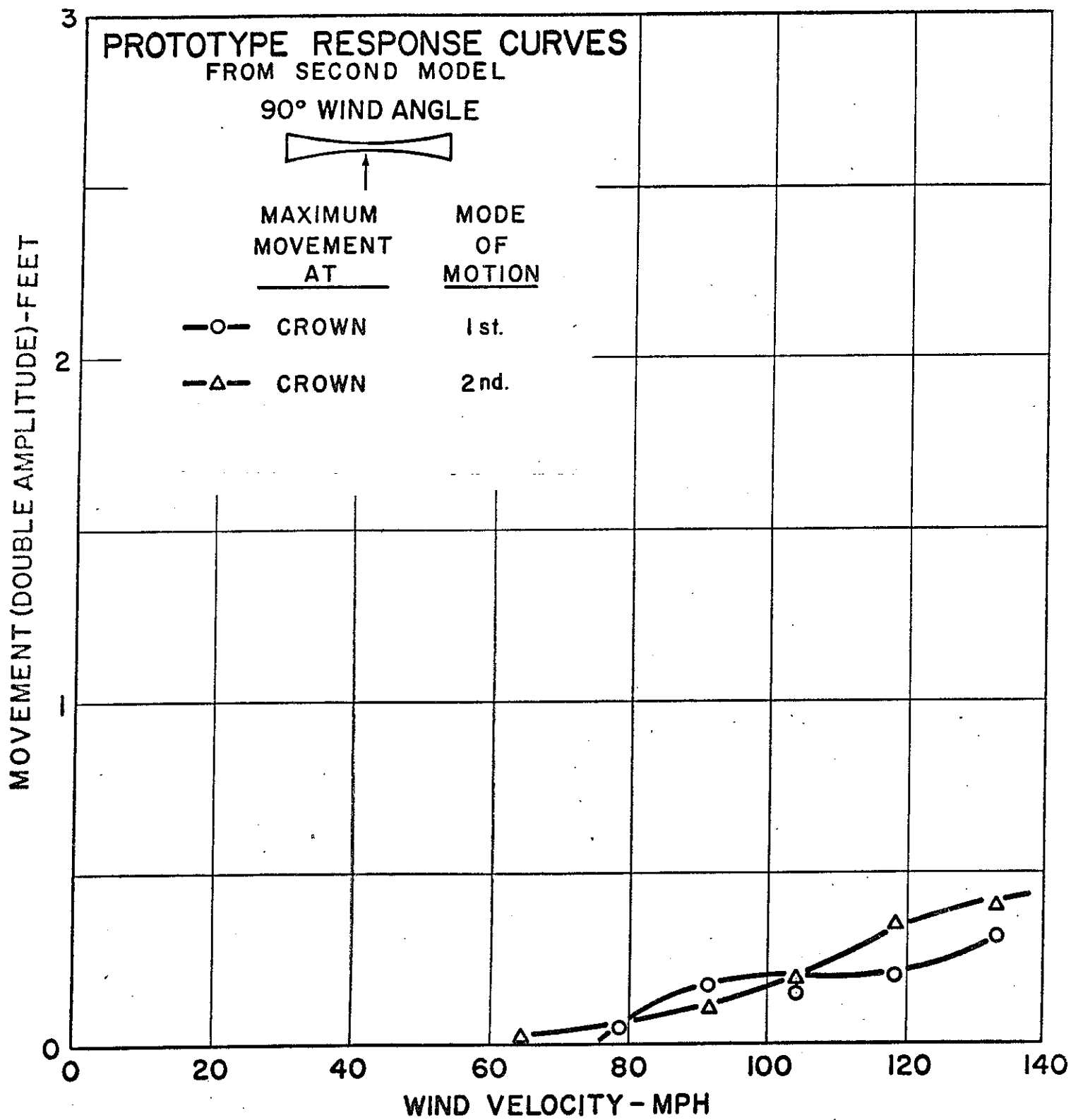


Fig. - 6

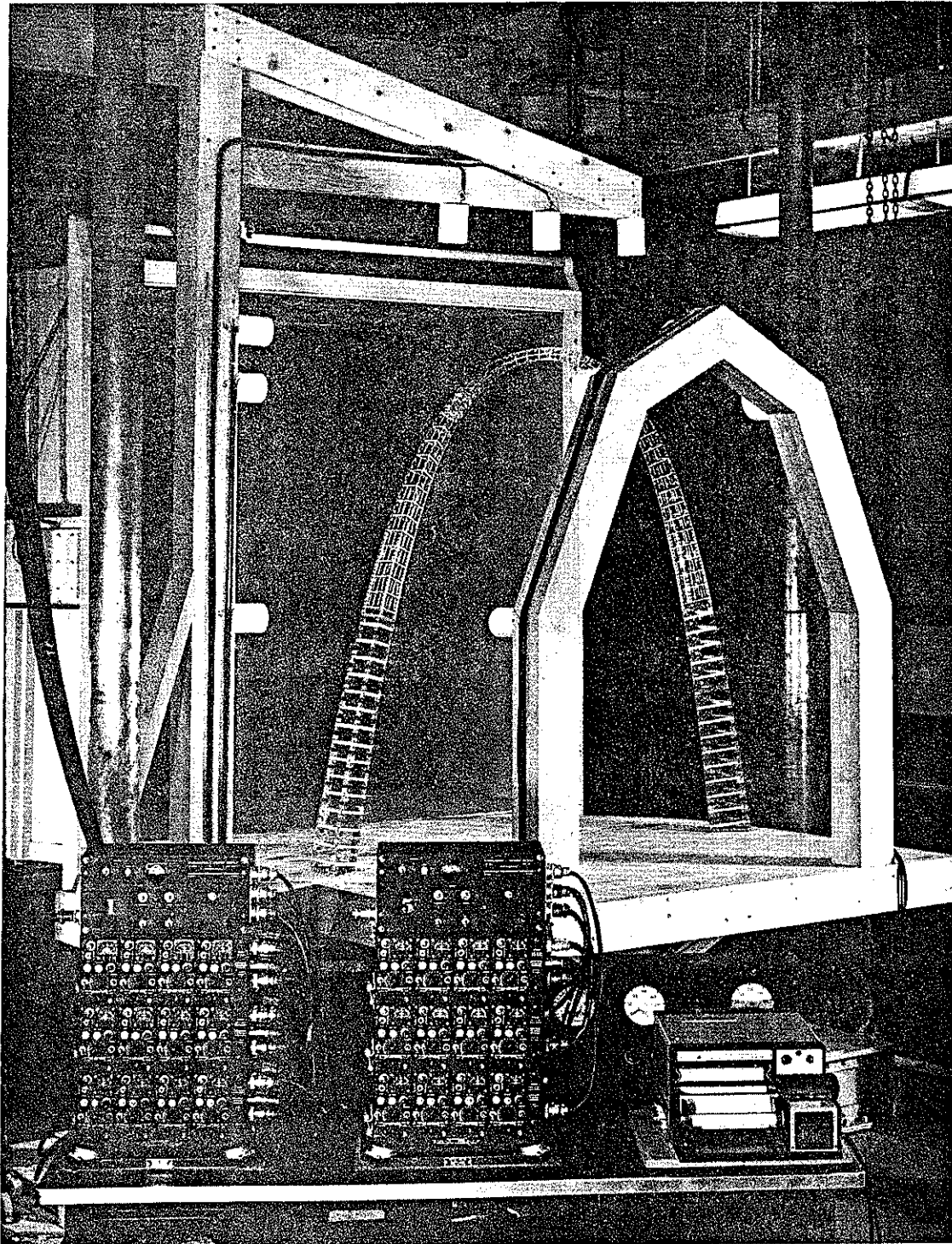


Figure 7. TEST SETUP - General view showing instrumentation and second model of arch oriented for 90° wind angle.

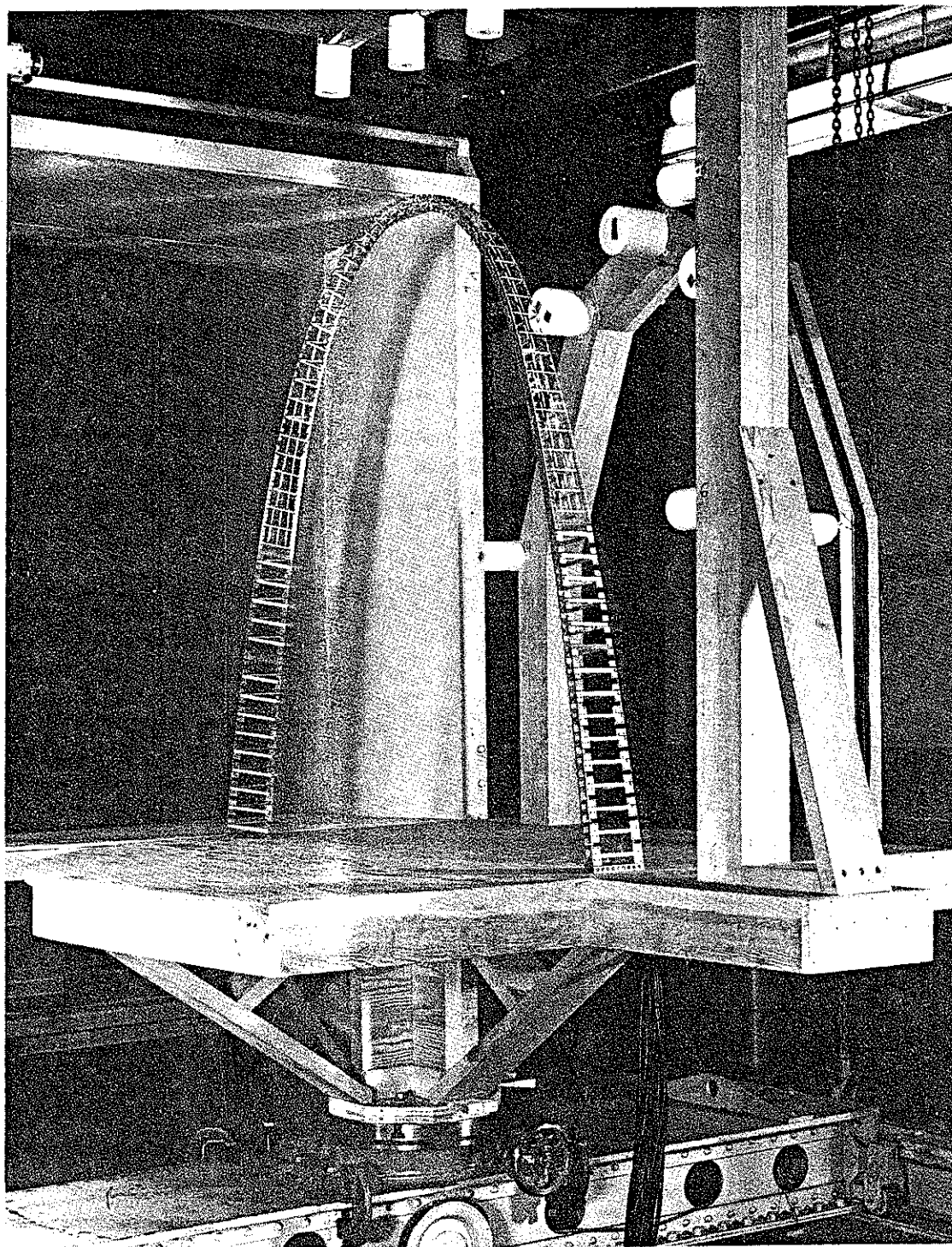


Figure 8. MOUNTING FOR ARCH MODEL - Second model of arch oriented for 0° wind angle. Note turntable for rotating platform, model and pickups as a unit.

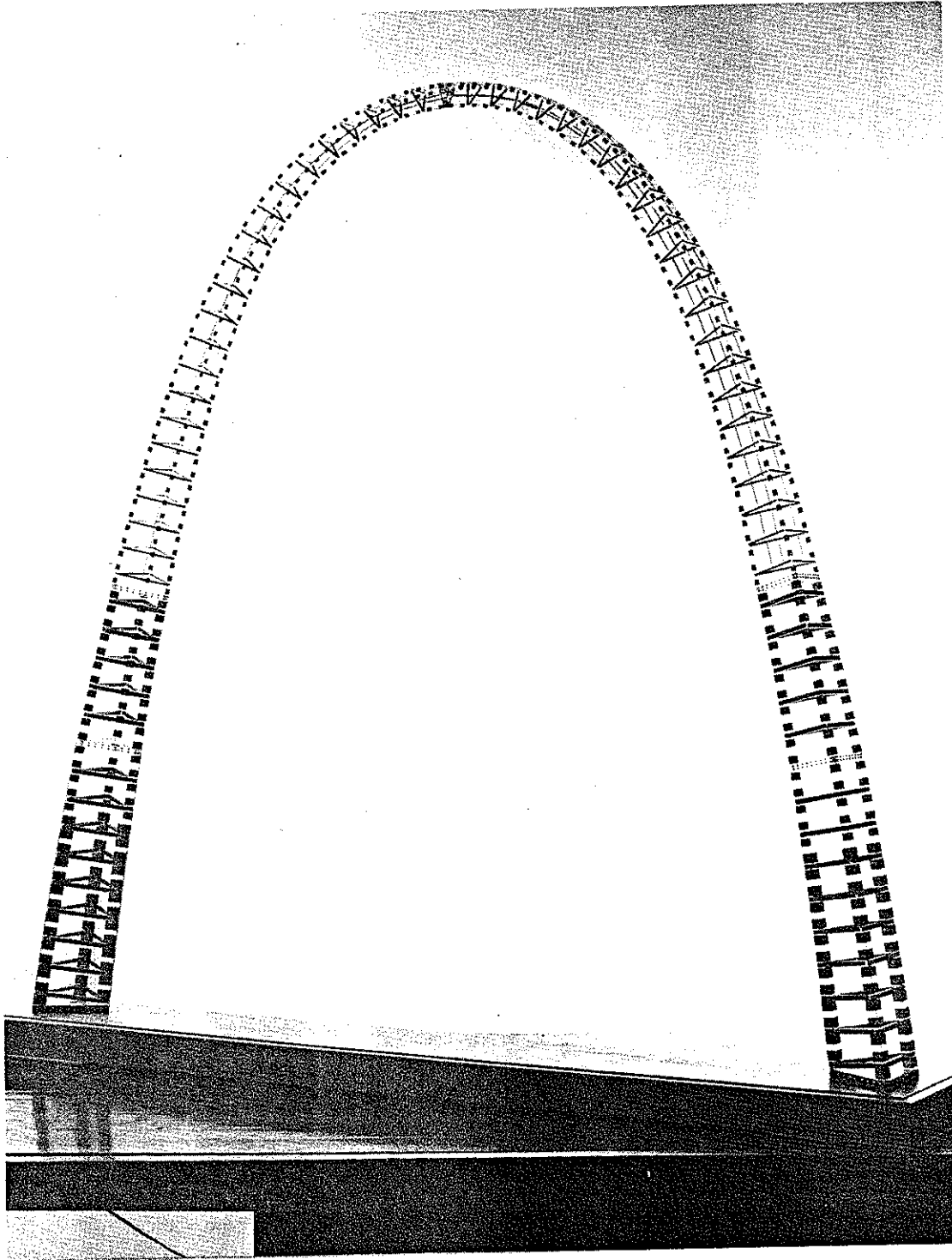
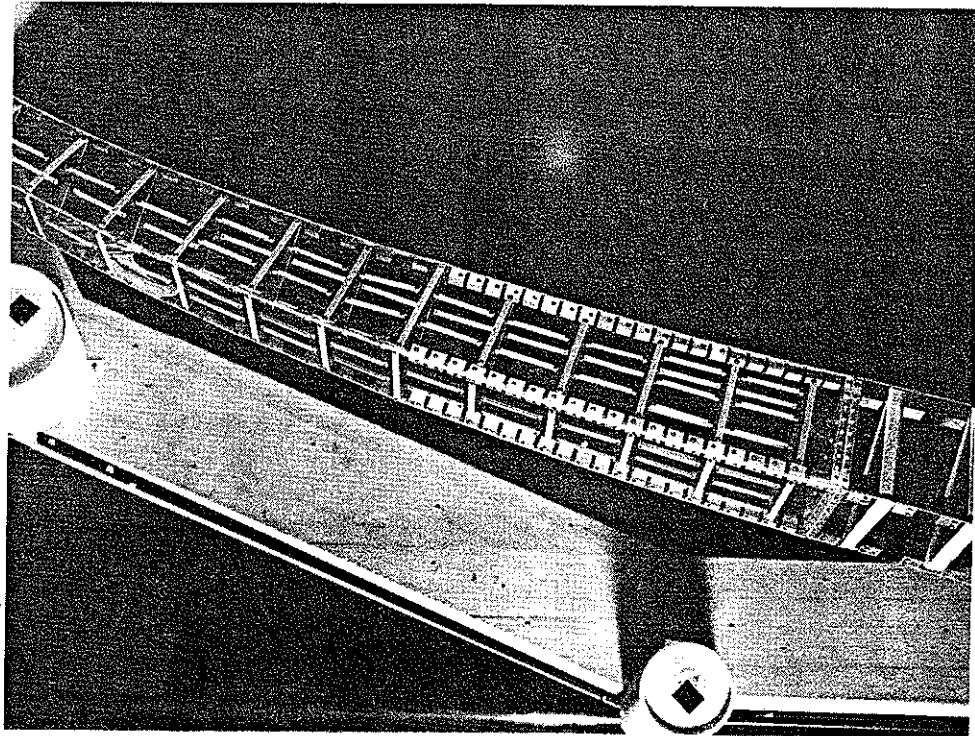
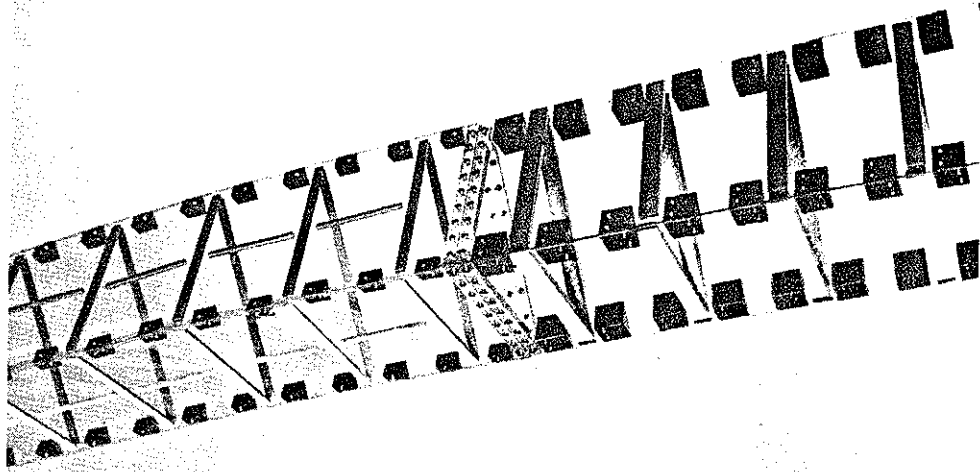


Figure 9. FIRST MODEL OF ARCH - Shown mounted on temporary fabrication base.

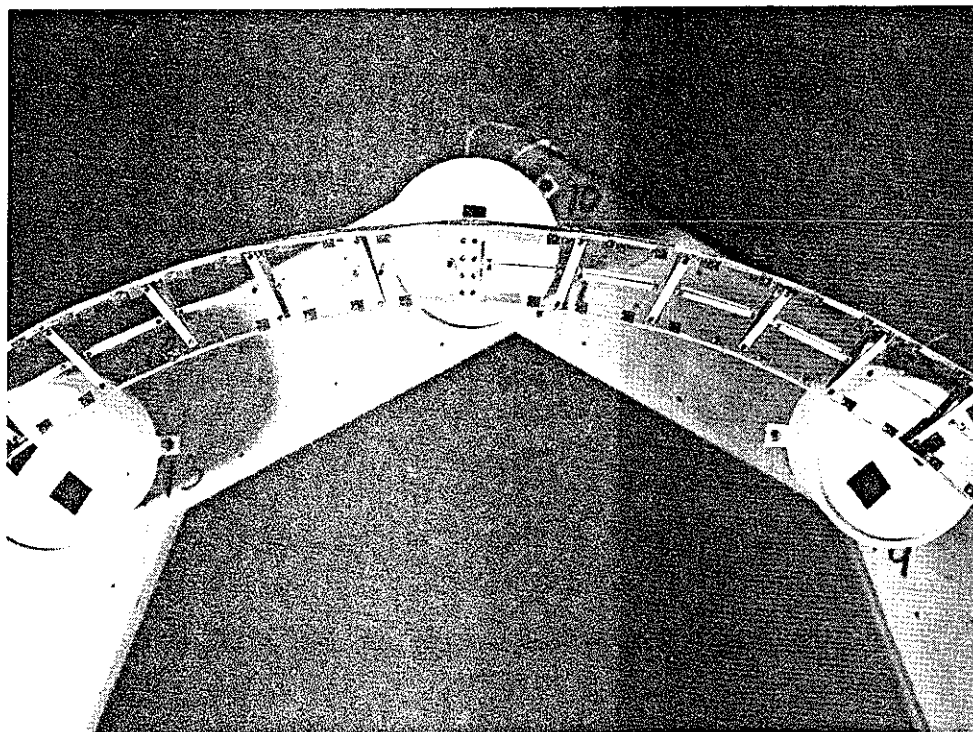


A. Station 48 to 26 (approx.) of second model.

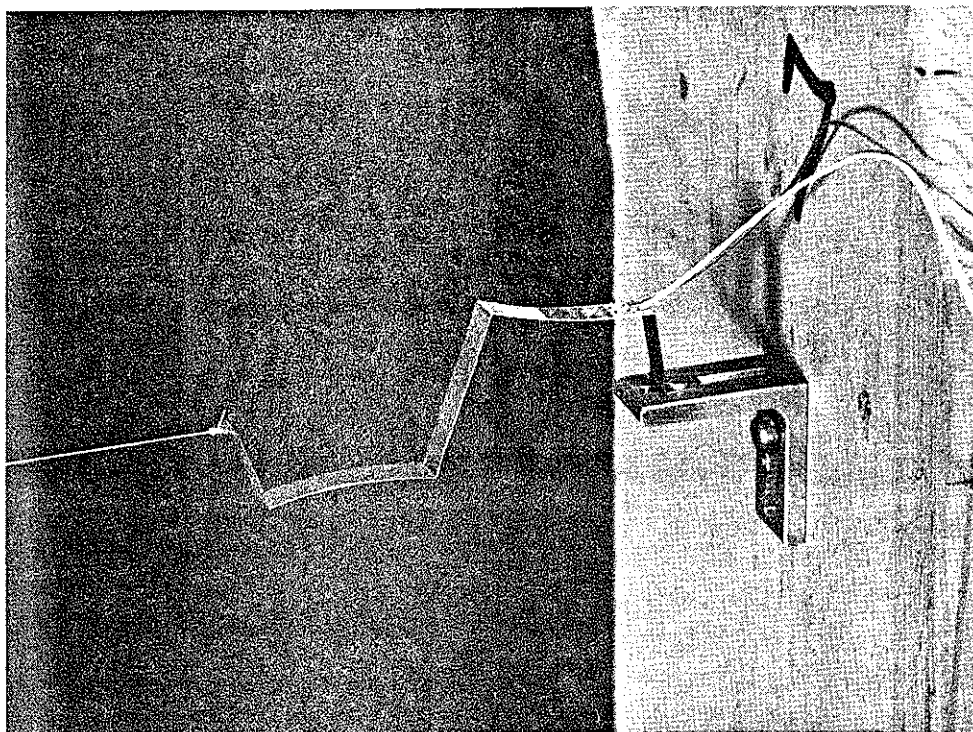


B. Station 51 to 40 (approx.) of first model.

Figure 10. SECTIONS OF ARCH



A. Crown section of second model.



B. Typical vibration pickup and mounting.

Figure 11. SECTION OF ARCH AND CLIP GAGE

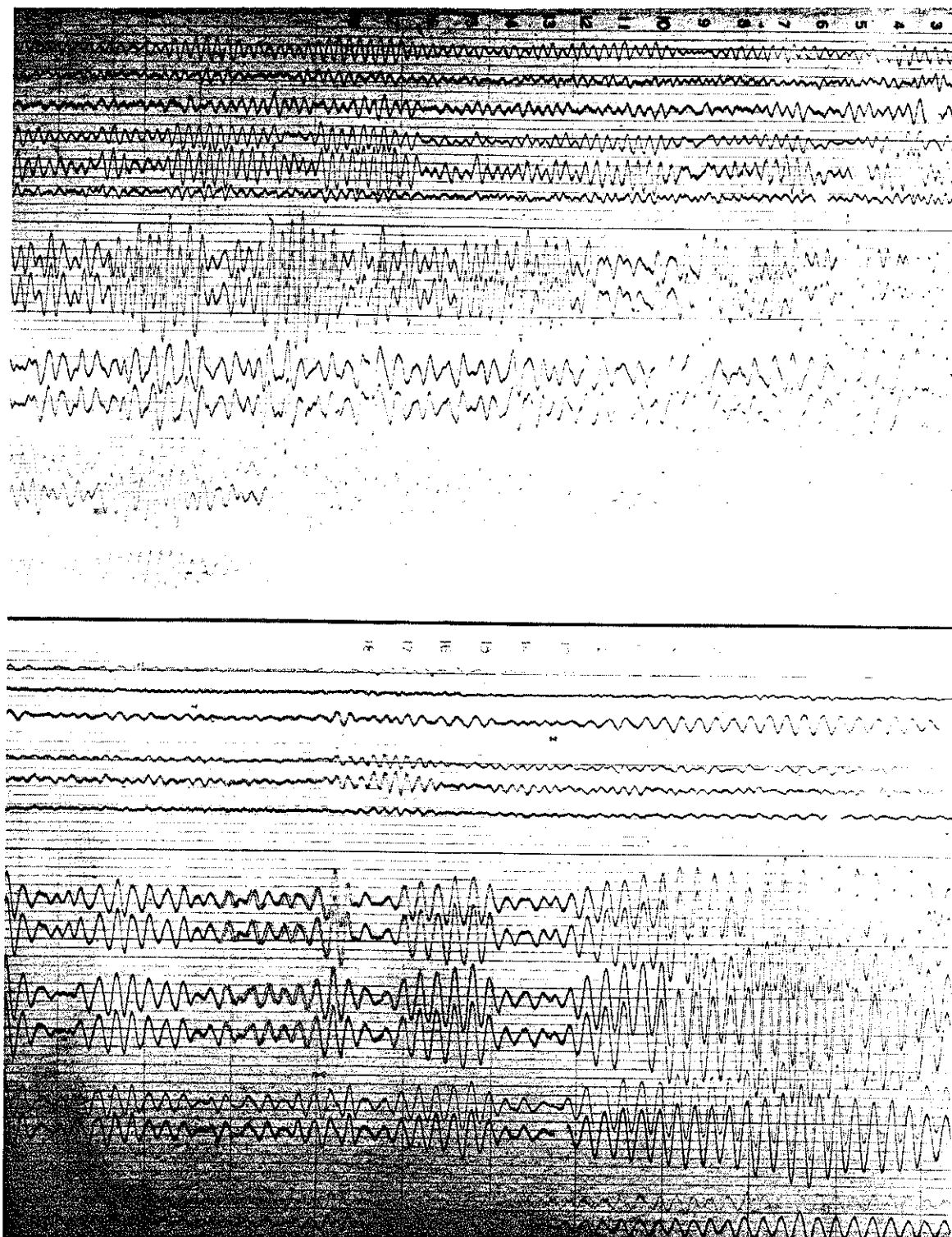


Figure 12. TYPICAL OSCILLOGRAPH RECORDS - For a 0° wind angle the top record shows movement in 120 mph prototype wind velocity, and bottom record is for 80 mph prototype wind velocity.

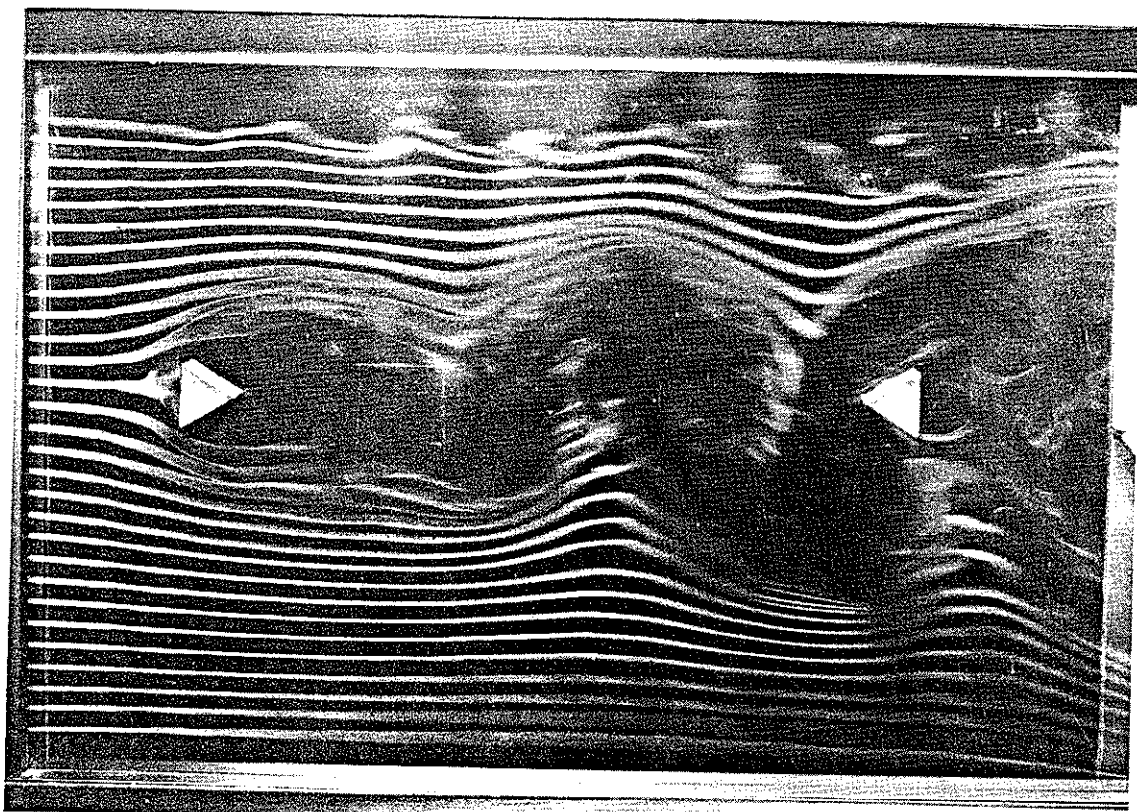


Figure 13. VORTEX STREET - Vortices formed by smoke streams flowing past a 1/300 scale model of arch section at station 21.

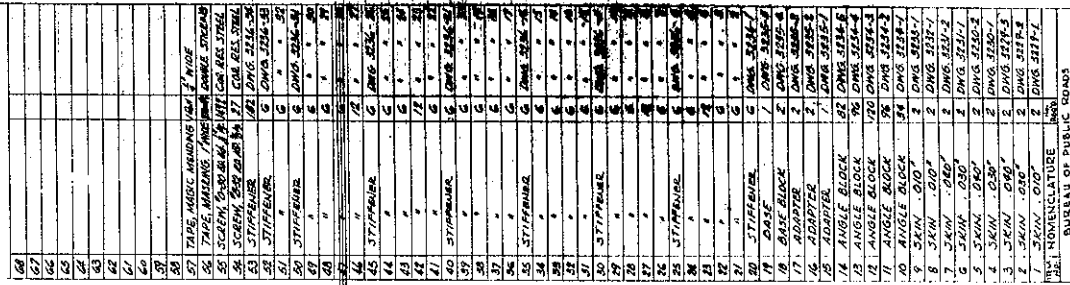
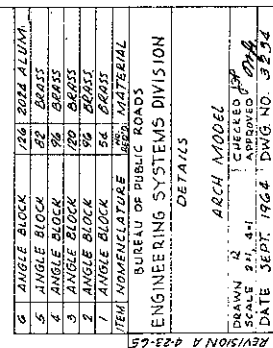
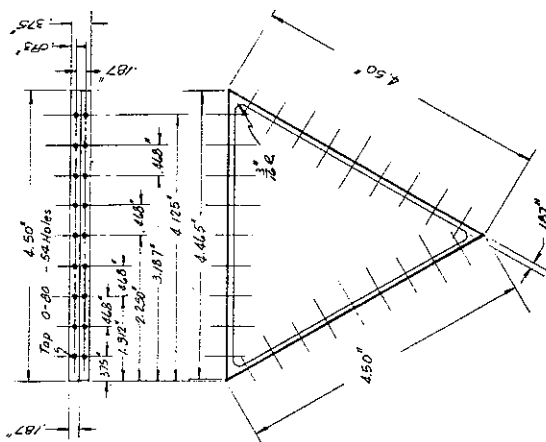


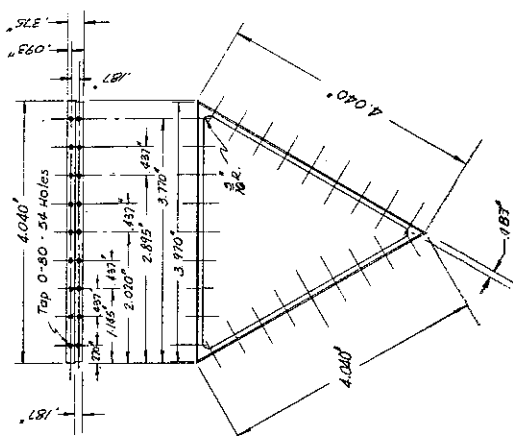
FIGURE 14. FIRST ARCH MODEL

ENGINEERING SYSTEMS DIVISION	
ASSEMBLY	
ARCH MODEL	
DRAWN R.	CHECKED <i>JS</i>
SCALE 1" = 20'-0" x 2"	APPROVED <i>MS</i>
DATE FEB. 1964	DWG. NO. 3337



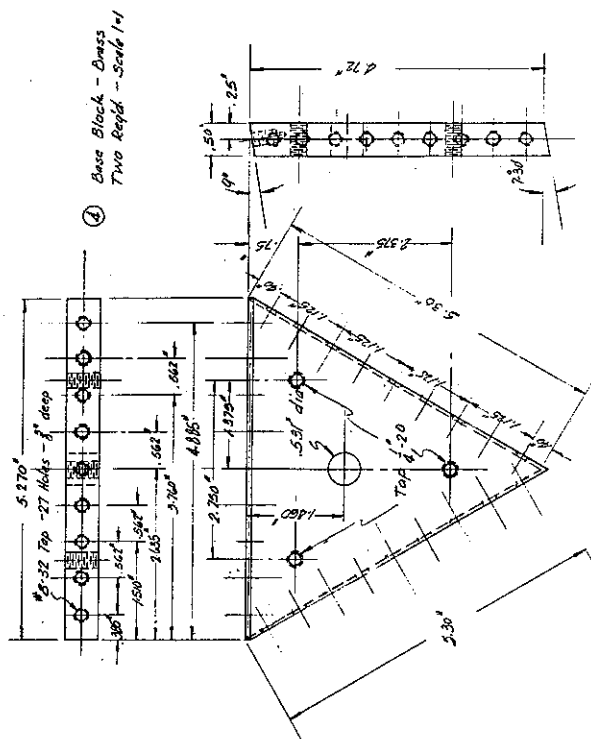


① Adapter - Lucite - One Reg'd. - Scale 1/4
Section O



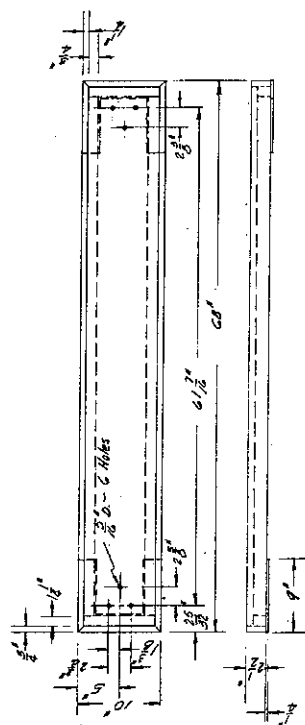
② Adapter - Lucite - Two Reg'd. - Scale 1/4
Section 4-5

③ Adapter - Lucite - Two Reg'd. - Scale 1/4
Section 54 1/2



④

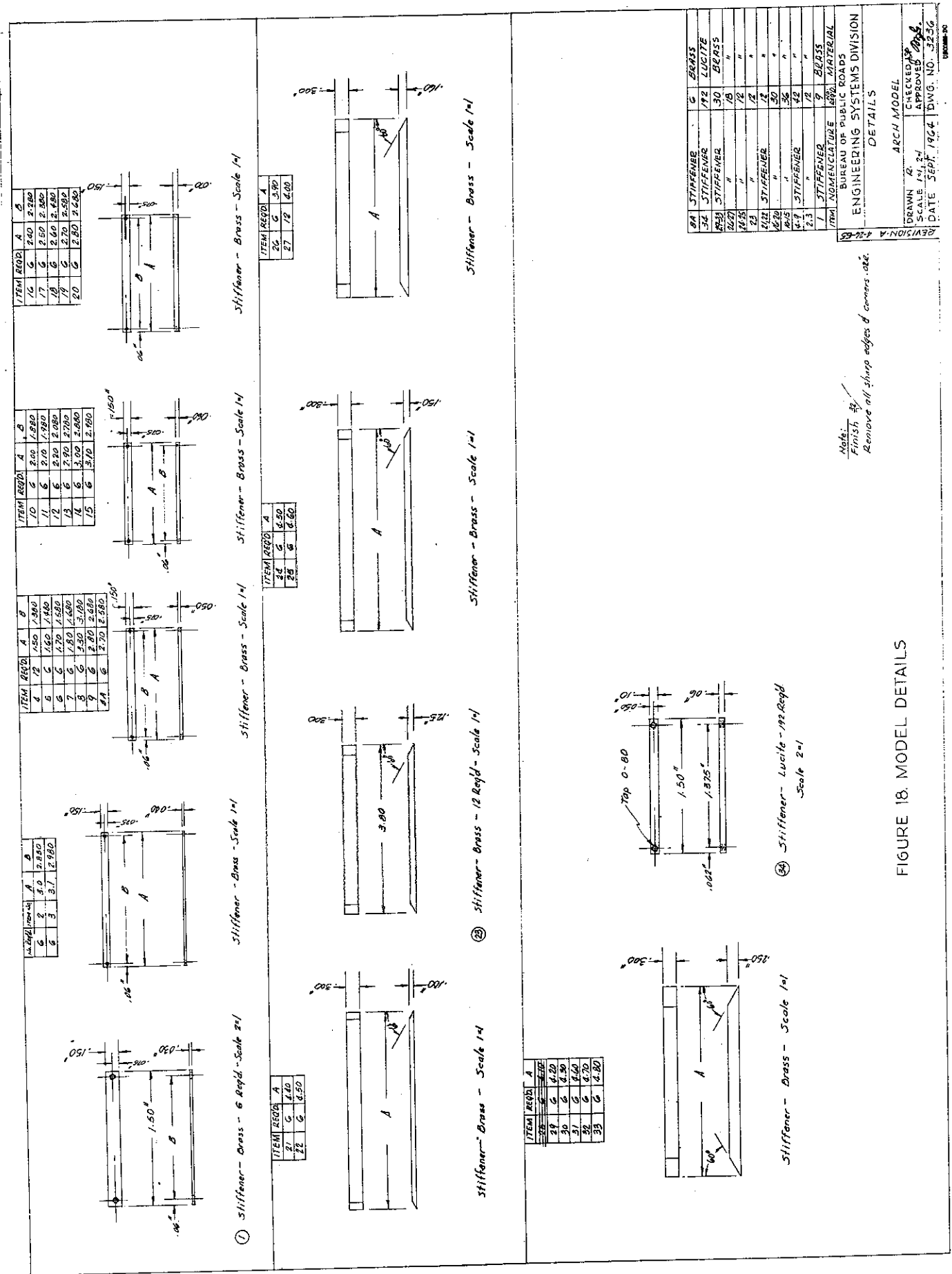
Base Block - Brass
Two Reg'd. - Scale 1/4



⑤ Base - Wood - One Reg'd. - Scale 1/4
Finish - Clear Sealer

ITEM	DESCRIPTION	QUANTITY
1	BASE	1
2	BASE BLOCK	2
3	ADAPTER	2
4	ADAPTER	2
5	ADAPTER	1
ITEM NOMENCLATURE		
BUREAU OF PUBLIC ROADS		
ENGINEERING SYSTEMS DIVISION		
DETAILS		
ASCH MODEL		
DRAWN	SCALE	CHECKED
1/1	1/1	APPROVED
DATE	SEPT. 1964	DWG. NO. 3235

FIGURE 17. MODEL DETAILS



APPENDIX A

FIELD STUDY OF PARTIALLY COMPLETED ARCH

by Robert F. Varney

April 1965

During the period April 7-13, strains and accelerations were recorded at two levels on the south leg of the arch in connection with the various movements induced by wind and construction operations. This leg of the arch had been erected to station 30 (extrados elevation 467.0) at the time of the field study. This report provides a description of the instrumentation and a discussion of the measured strain and acceleration values.

INSTRUMENTATION

a. Strain Gages

Access was provided by scaffolding to the 3 interior corners and the extrados face inside the arch at station 42, about 25' above the top level of the concrete filled portion of the arch. Bonded wire strain gages were installed on the center of each of the three corner plates at this station and parallel to the vertical corner lines. A 45-degree strain gage rosette was installed on the extrados face midway between the corners at the same station with the center gage oriented vertically. All gages were applied on clean bare metal with appropriate preparation and care and were supplemented by temperature compensating gages on unstressed steel blocks in intimate contact with the side of the structure near each active gage.

b. Accelerometers

Statham accelerometers were clamped to the structure near the strain gages at station 42 in each of the two extrados corners. Pairs of these instruments were mounted in a horizontal plane at both corners to record separately the simultaneous north-south and east-west accelerations.

At the top of the structure (station 30) two Statham accelerometers were clamped to the backing plate between the inner and outer surfaces on the extrados to register separately the simultaneous east-west and north-south accelerations in a horizontal plane at that level.

All transducers were energized by a carrier voltage generated in dynamic strain amplification equipment installed on the stair landing at station 44 and connected directly to each transducer by 4-conductor cables. The output of each transducer was transmitted through the same cable to galvanometers incorporated in a recording oscillograph located with the other equipment.

LOADING

The installation of transducers at station 42 was completed late in the afternoon of April 7 while a segment of the arch was being lifted into place. A few trial records were taken at this time although the telephone circuit between the recorder and the top had not been installed. On April 8 a north-south oriented accelerometer was installed near the west corner of the extrados at station 30 and communications were established by phone from the recorder to the top of the structure. Vibration records were then taken at intervals and the related vibration-inducing activity was recorded as reported by an observer at the top. A strong wind estimated at 30-35 mph was blowing, first from the south and shifting to the west. Records were taken of the effects of the wind on the structure. On April 9, an east-west oriented accelerometer was installed at the top near the east corner of the extrados and records were taken at intervals as before. Winds were light this day. The contractor's hoist operator cooperated by inducing some vibrations of the structure by manipulating the movement of the boom in a manner known to him to induce a vibration of the structure. These motions were induced with the boom heading in a northeasterly direction. Records were taken of the resulting vibrations each time.

On April 12 the east-west oriented accelerometer was mounted alongside the north-south oriented one at the west corner of the extrados. Winds of 20-30 mph were blowing from the south and west and records of the effect were taken. At our request the hoist operator induced additional vibrations in the structure which were recorded first with the boom facing north and then to the northwest. On April 13 another segment of the arch was lifted into place with only a slight wind blowing and a record was made of the resultant strains during the full sequence of operations. This concluded the field observations.

Each day the strain and accelerometer circuits were calibrated electrically. The accelerometers were calibrated mechanically at the site once, and again in our laboratory following the return from the field.

RESULTS

The accelerometer installations were designed to indicate vector displacement, frequency and damping associated with any induced vibrations. The measurements were made to study the stresses developed as a result of the vibrations as well as incremental non-vibratory stresses resulting from the raising and placing of an additional arch segment. Since no controlled experimental procedure was possible, the results of the field study of the arch are reported in narrative form.

During the five days on which records were taken, the most frequently observed phenomenon was the accelerometer response at a frequency of vibration of 0.86 cycles per second in the north-south direction and 0.89 cycles per second in the east-west direction. These values correspond closely to the predicted fundamental mode frequency. Wind induced movement was always at this frequency and was distinctly perceptible to observers at the top and at station 42 on both windy days. The maximum computed amplitude of motion was ± 0.03 inch at the top in a north-south direction with a simultaneous in-phase movement of ± 0.02 inch at station 42. The corresponding maximum strain in the intrados was a barely distinguishable 25 psi. At times the east-west motion was greater than that in the north-south direction but never reached the same maximum amplitude described above. The top motion amplitude was approximately twice that of station 42 for the east-west motion. The relative magnitude and direction of the movements indicate that the structure was responding in the fundamental mode of a cantilever bending from the base.

The other occasion of movement in the structure was caused by the intermittent braking of the moving hoist boom either during working operations or at our request. A distinct and somewhat violent shudder was perceptible to observers and disconcerting to workmen at both elevations in the structure when the boom was braked. Analysis of the top accelerometer records of this action revealed that the predominant response consisted of an initial impulse varying around 3 cycles per second and shifting to the 0.86 cycle frequency after three to five seconds if winds were strong and after 10 seconds or more if winds were light. The response was highly directional. When the boom was oriented in a north-south direction, the initial response in the east-west oriented accelerometers was slight. When the boom was facing the northeast, the normal raising and lowering position, or the northwest, the effects were noticeable in both the north-south and the east-west instruments. The accelerometers at station 42 did not respond at 3 cps during braking, but vibrated at 1.3 cps briefly. This would indicate the 3 cps vibration is a forced frequency from the hoist mechanism. The first attempt to shake the structure on April 8 was the most violent and the succeeding attempts were more restrained. The "violent" shake developed a peak stress of only ± 300 psi in the intrados. Thereafter about ± 100 psi was the maximum attained by braking the boom. The simultaneous stresses in the extrados corners and face were in tension and about half the magnitude of the respective intrados stress. Based on projections from the wind effects, the 300 psi stress is equivalent to a ± 0.36 inch peak displacement at the top at a 0.86 cps vibration, and the 100 psi stress is equivalent to a ± 0.12 inch motion. These values are considered more representative of the magnitude of motion observed than that which can be obtained by direct integration of the high frequency accelerometer responses.

The damping of the induced vibration in the structure is a difficult characteristic to measure and define. Wind vibration records were not useful for damping studies since the motion of the structure could not decay due to constant energy feedback from successive gusts. Large amplitudes of vibration in the fundamental mode were never provided by the boom-induced

vibrations since the braking of the boom movement produced a 3 cps vibration which changed to the fundamental mode only after the decay curve was already in the asymptotic region. The logarithmic decrement in this region was determined to be 0.085. The vibration decay curve often resembled one of pure friction damping with logarithmic decrement values determined from samples of many records in this area ranging from 0.13 to 0.19 in the fundamental mode with peak vibration ordinates falling on a linear descending slope. Logarithmic decrement values taken near the beginning of the 3 cps vibration decay region gave values as high as 0.27. These latter values may not be significant since the 3 cps vibrations are apparently forced by the action of the hoist mechanism. Peak stress excursions (± 300 psi) caused by the boom braking action provided no useful damping data.

A sequence of static strain readings was made during the time a segment of the arch (reported to weigh 15 tons) was being raised from the ground and put into place at the top between 8:00 a.m. and 10:00 a.m. No abrupt change of stress was noted as the segment was picked off the ground. It was noted that stresses began to change gradually with time as the lift was made. After the lift was completed and the lift lines to the segment had been released, the stresses continued to change at a similar rate. The stresses recorded may therefore represent thermal stresses induced in the structure by the heating effect of the sun rather than load-induced stresses.

SUMMARY

The live loads applied to the prototype were quite limited due to the relatively low wind velocities experienced and the caution which had to be exercised in manipulating the hoist. The resultant measured responses are therefore quite small. Because of the resulting wide disparity between the amplitudes of induced motion of the model and of the prototype, direct comparisons of any values except fundamental frequency are not appropriate. The fundamental frequency correlation was excellent, 0.89 cps in the prototype compared with 0.91 cps in the model. While not final or conclusive, the damping and stress data provided for the prototype do give some insight into the probable behavior of the completed structure.

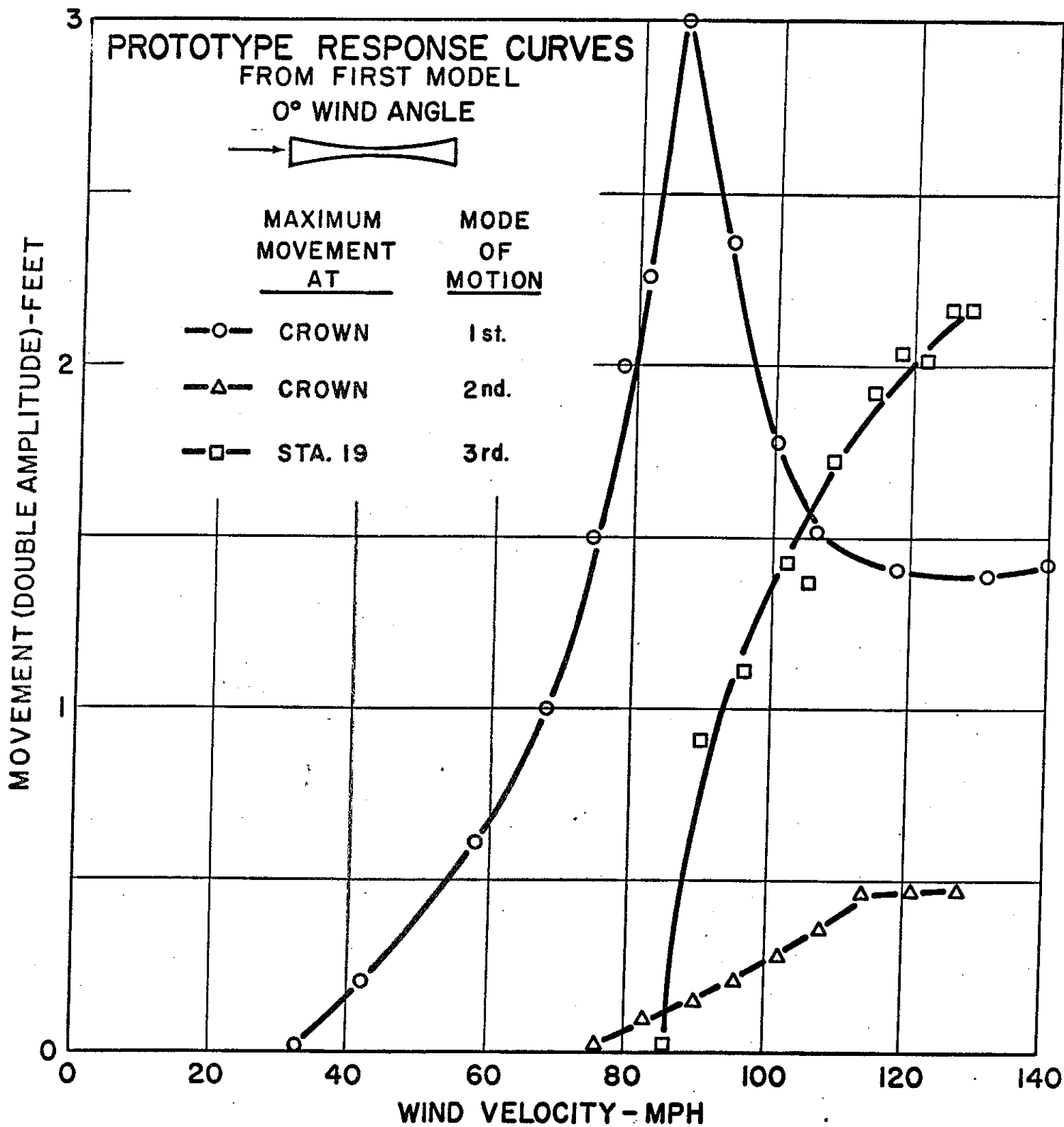


Fig. B-1

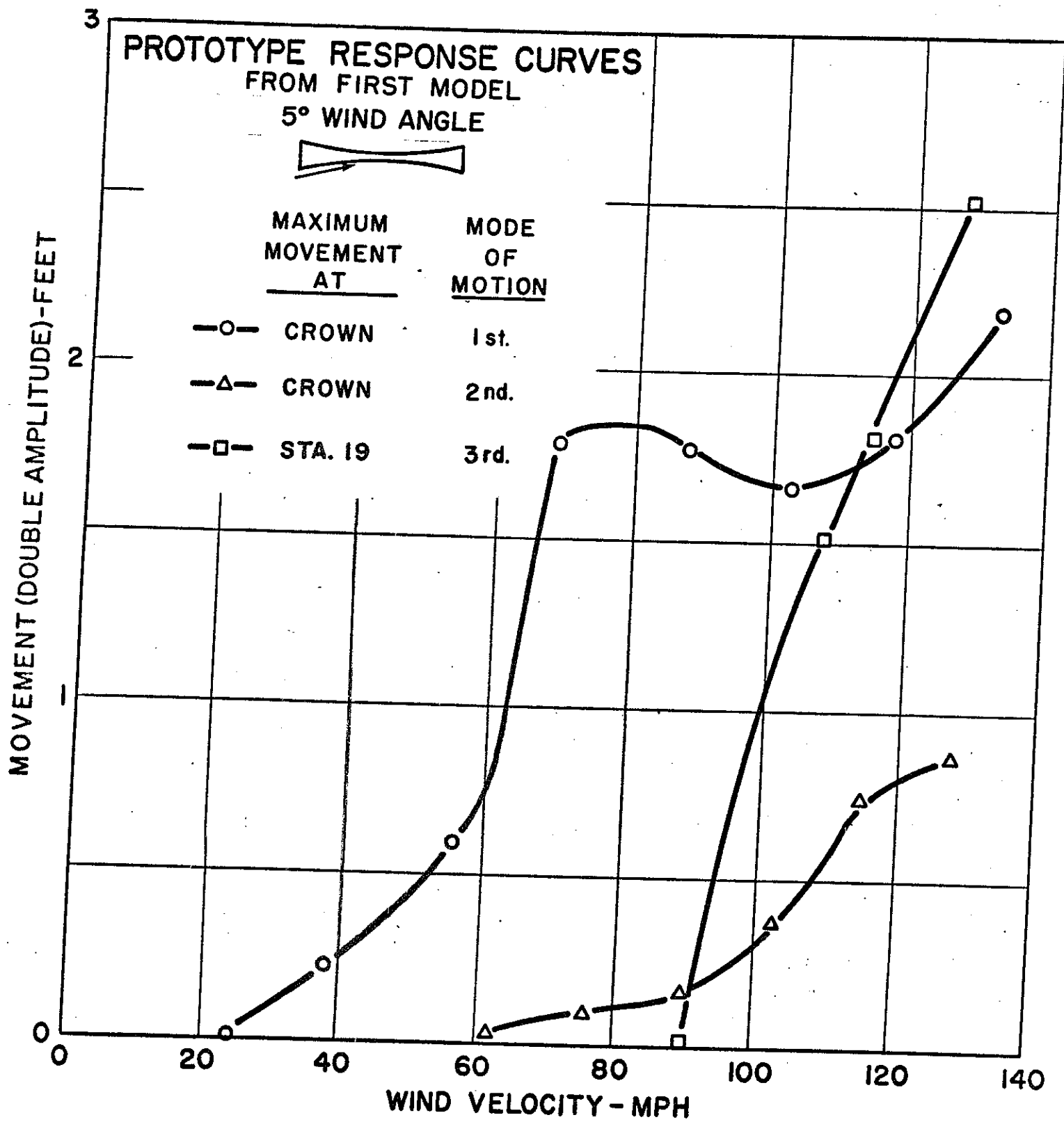


Fig. B-2

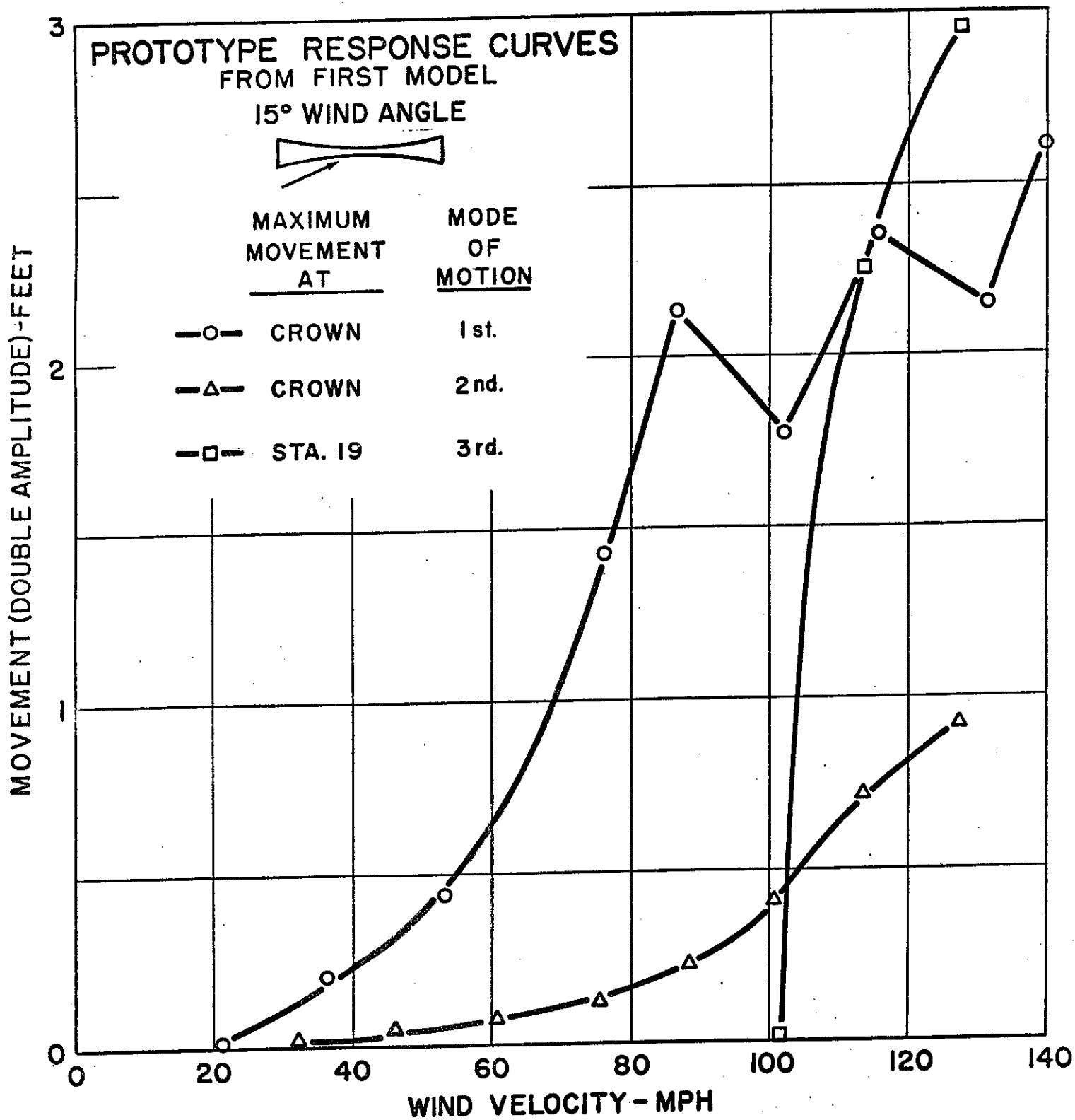


Fig. B-3

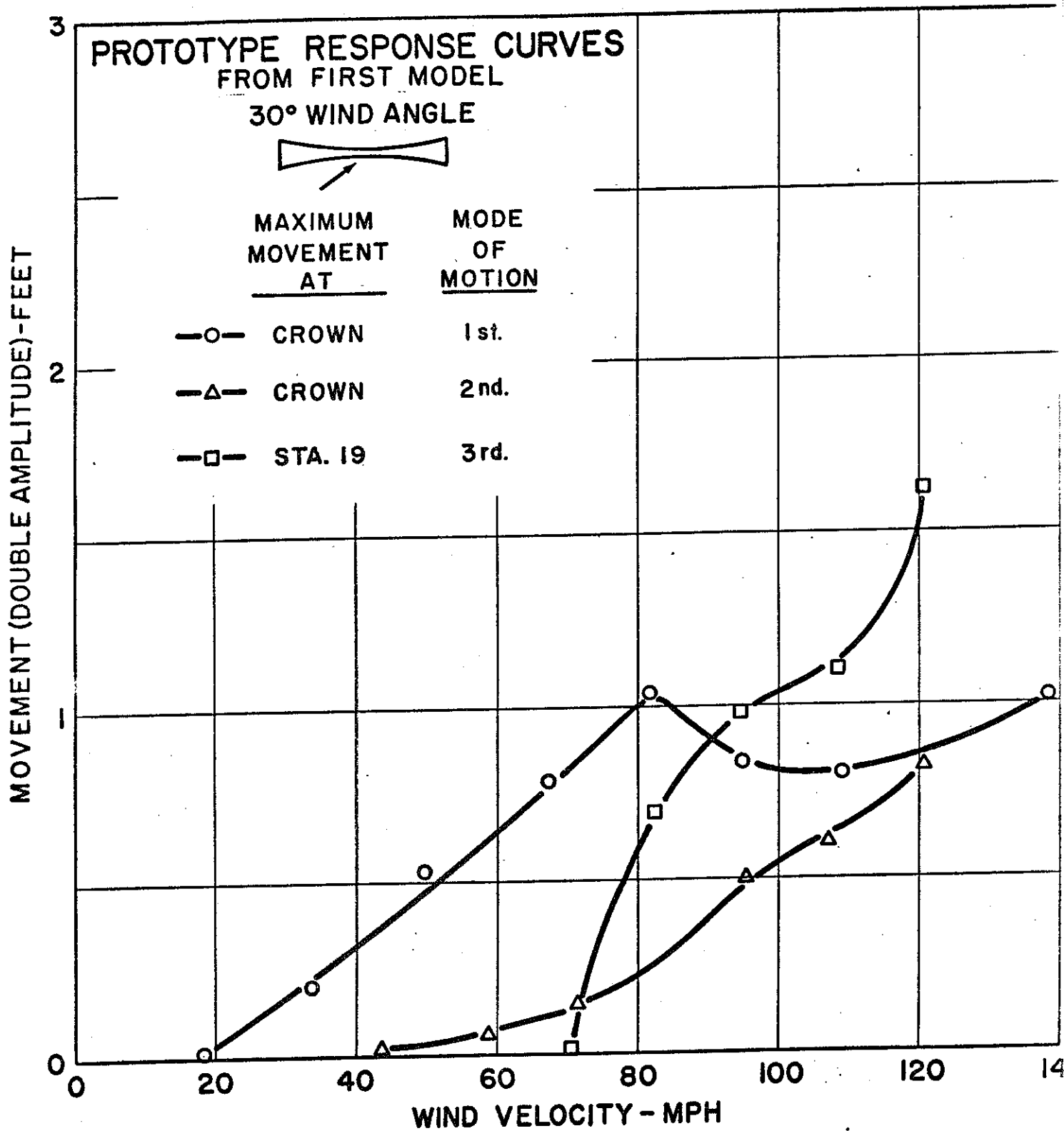


Fig. B-4

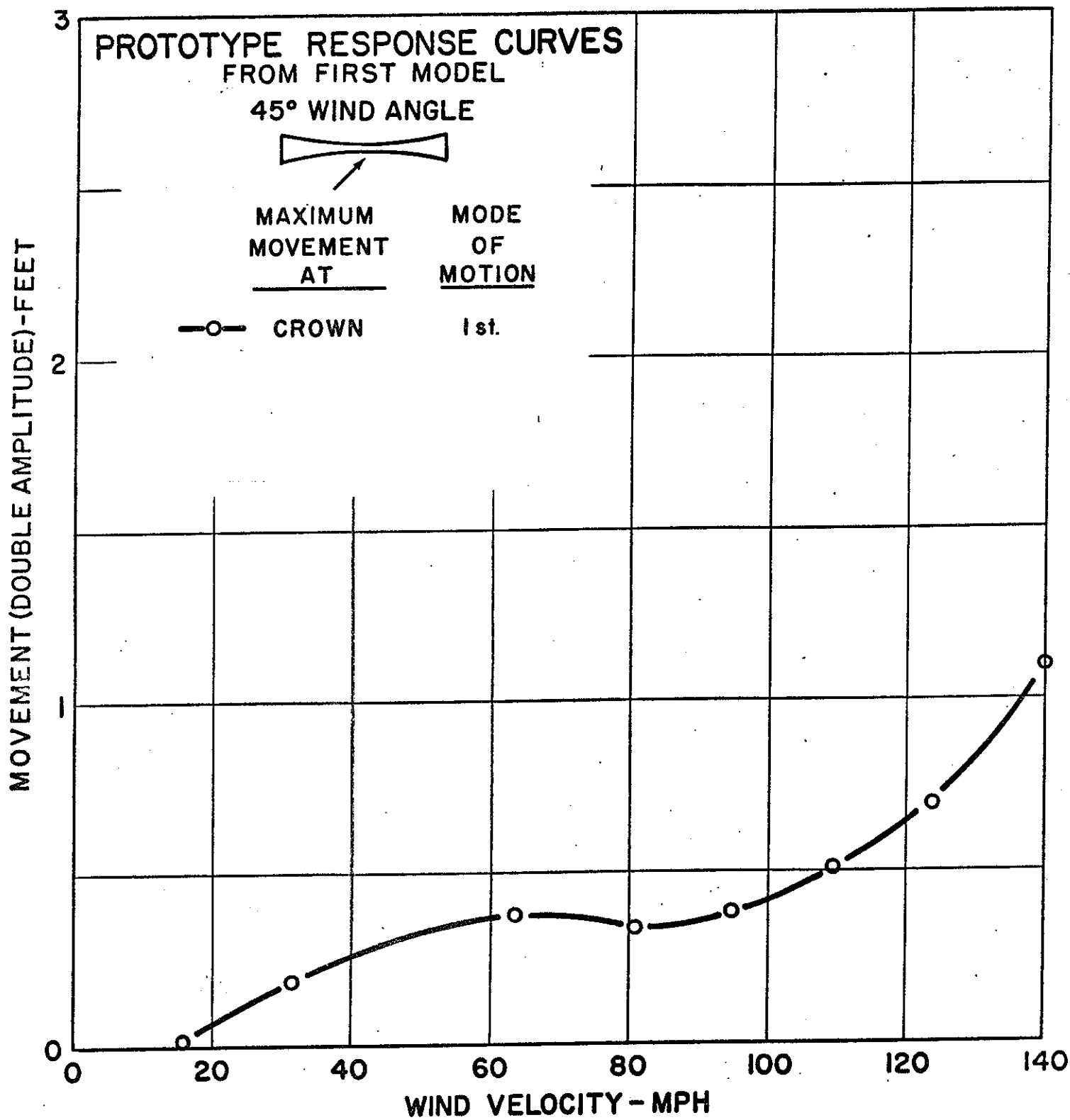


Fig. B-5

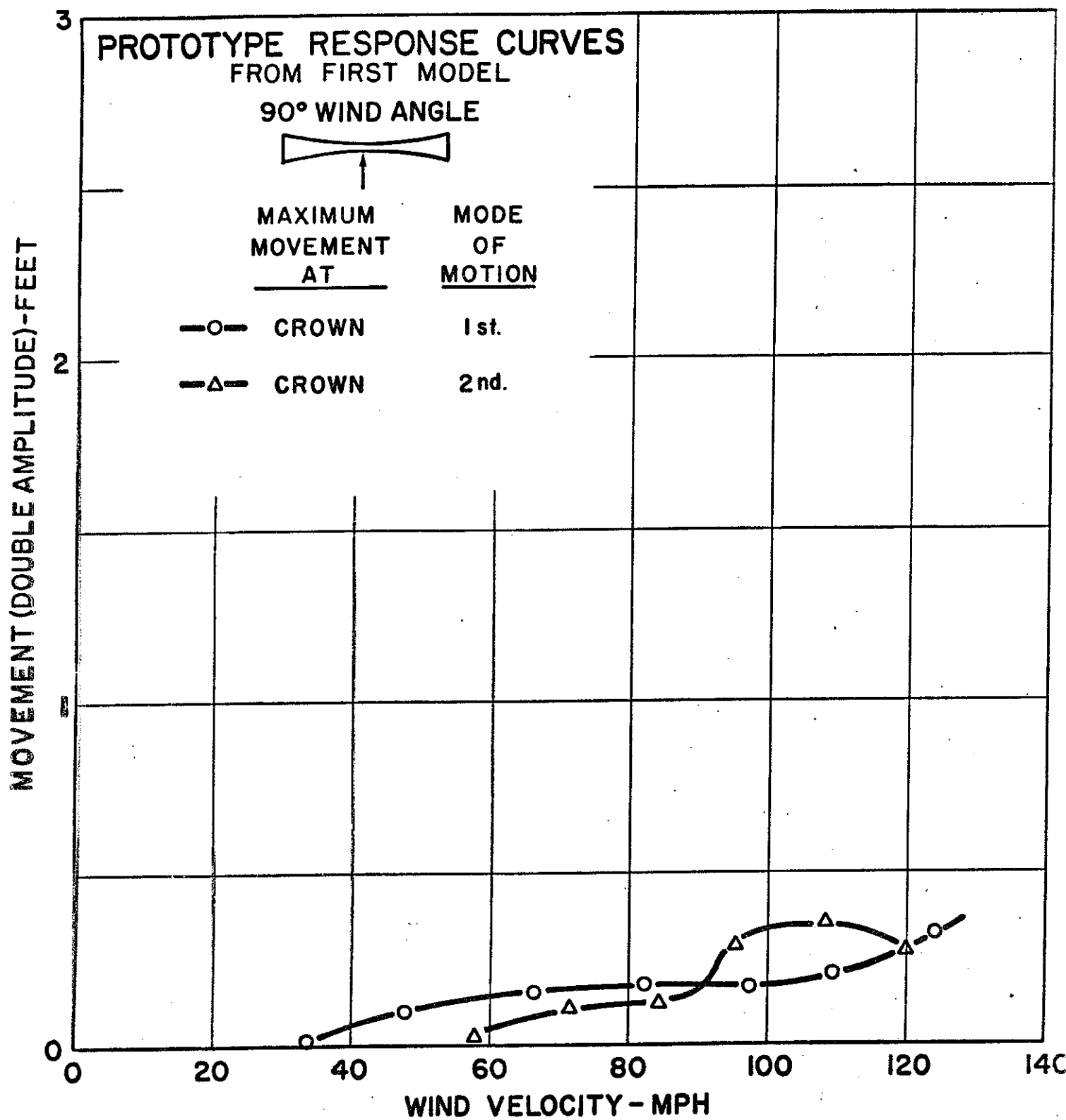


Fig. B-6

Appendix D – Interior Temperature and Relative Humidity Monitoring

Via Email

December 3, 2008

Dan M. Worth
Senior Principal
Bahr Vermeer Haecker Architects
440 N 8th Street, Suite 100
Lincoln, NE 68508

Re: Gateway Arch HSR Preliminary Temperature and Relative Humidity Monitoring
WJE No. 2008.3721.1

Dear Mr. Worth:

During the initial staining study completed in 2006 by the BVH/WJE team recommendations were made to design a monitoring program in order to measure temperature and relative humidity (RH). As part of the present HSR work, we decided to gather some initial temperature and RH data, by installing four Hobo Pro data loggers, three in various locations inside the south leg of the Gateway Arch, as well as one on the exterior.

The data loggers are electronic instruments that cyclically record temperature and relative humidity over time. Typically, data loggers are small, battery-powered devices that are equipped with a microprocessor, data storage and sensor. The data loggers installed at the Gateway Arch are rectangular and are approximately 2.4"x1.9"x0.8" See Figure 1 for appearance and approximate size of Hobo loggers.



Figure 1. Appearance and approximate size of Hobo data loggers installed at the Gateway Arch.

Setting up and initializing the HOBO Pro loggers required the connection to a personal laptop with the BoxCar software and selecting the appropriate logging parameters (including sampling intervals and start time). The loggers began collecting data at 9a.m. on Thursday November 20, 2008 and are programmed to collect readings of the temperature and RH every 1.5 hours, thus 16 readings will be collected for each day. By taking readings every 1.5 hours the logger can hold up to nearly a year's worth of data. Upon completion of the setup, the loggers were initiated and deployed in the desired locations, as indicated in the chart below. The logger records each measurement (Temperature and RH) and stores it in memory along with the time and date. By collecting the temperature and relative humidity (RH) readings, the dew point measurement can be calculated, and displayed in the spreadsheet and graphically depicted with the measured parameters.

The following chart indicates the logger number, location of placement, as well as the photos that were taken of each logger at installation on Thursday November 20, 2008. These few loggers were installed to establish some baseline temperature, dew point and RH data for the Gateway Arch, and also to determine if significant variances exist in this data, because of the relative location in the arch, as well as compare with the exterior values.

Hobo #	Interior/Exterior	Location	Figures
1	Exterior	Outlet box lightpole #38	2
2	Interior	Base of south leg in foundation	3
3	Interior	Top of south leg on backside of stair beam #14	4
4	Interior	South leg at transition between steel and conc. #47	5

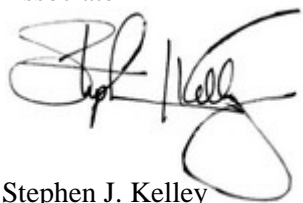
As part of this rough monitoring program, we respectfully request to have access to the south leg when needed to collect the data, as the information cannot be collected remotely. We also would appreciate if the maintenance staff could keep a log of dates and times when condensation is experienced inside the Arch legs, so we can correlate this information with the temperature and RH data we have collected.

Sincerely,

WISS, JANNEY, ELSTNER ASSOCIATES, INC.



Rachel L. Will
Associate II



Stephen J. Kelley
Principal



Figure 2. Monitor #1 is located in the electrical outlet box on light pole #38, located on the path leading to the south entrance.



Figure 3. Monitor #, as indicated by the arrow, is located just to the north above tram loading area in the foundation of the South Leg.



Figure 4. Monitor #3 located in the upper portion of the South leg on the backside of a beam supporting the landing of the stairs at the base of the spiral stairs. (Located around panel #14)

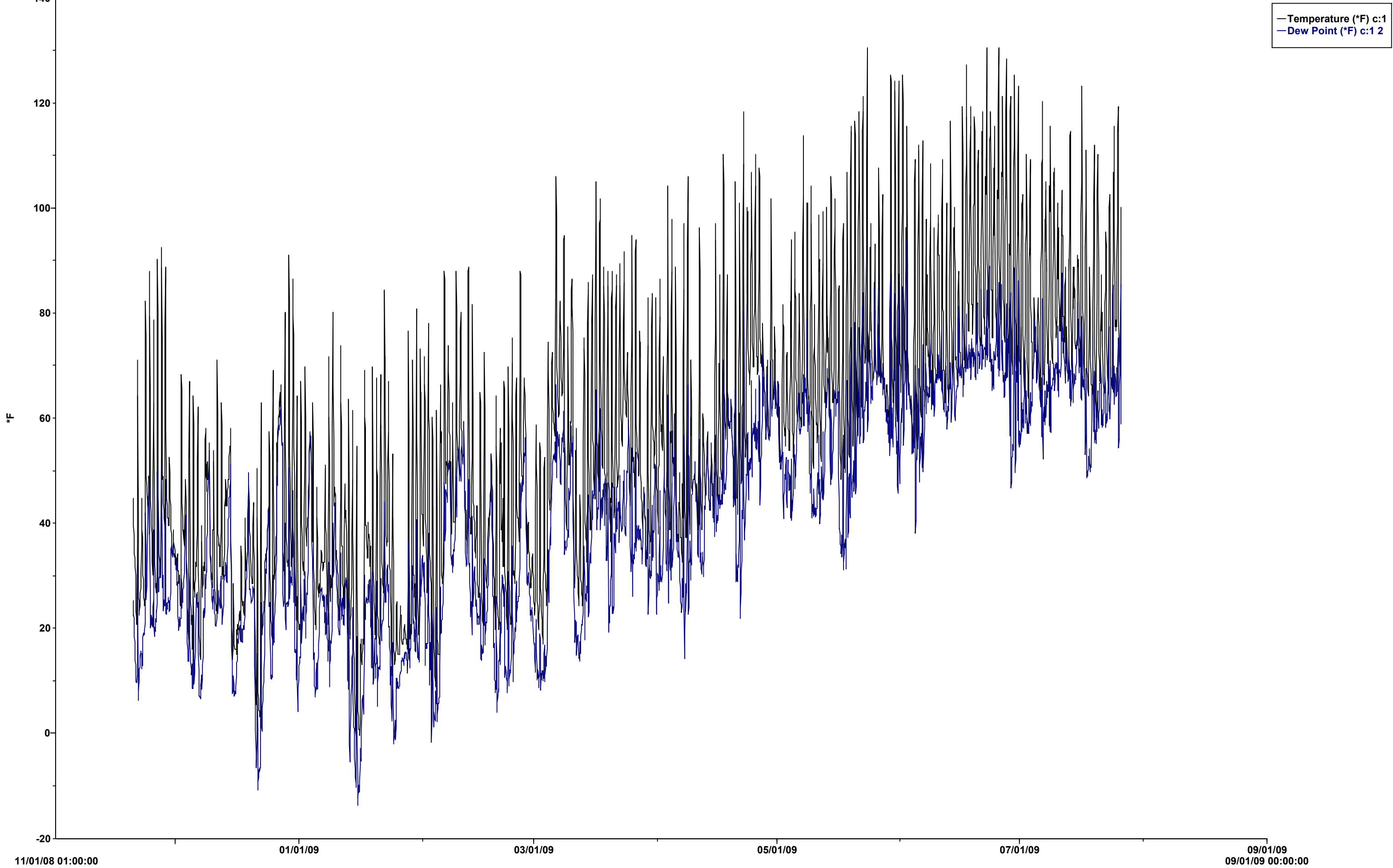


Figure 5. Monitor #4 located at the transition between steel and concrete fill of the South leg on a framing beam. (Located around panel #46)

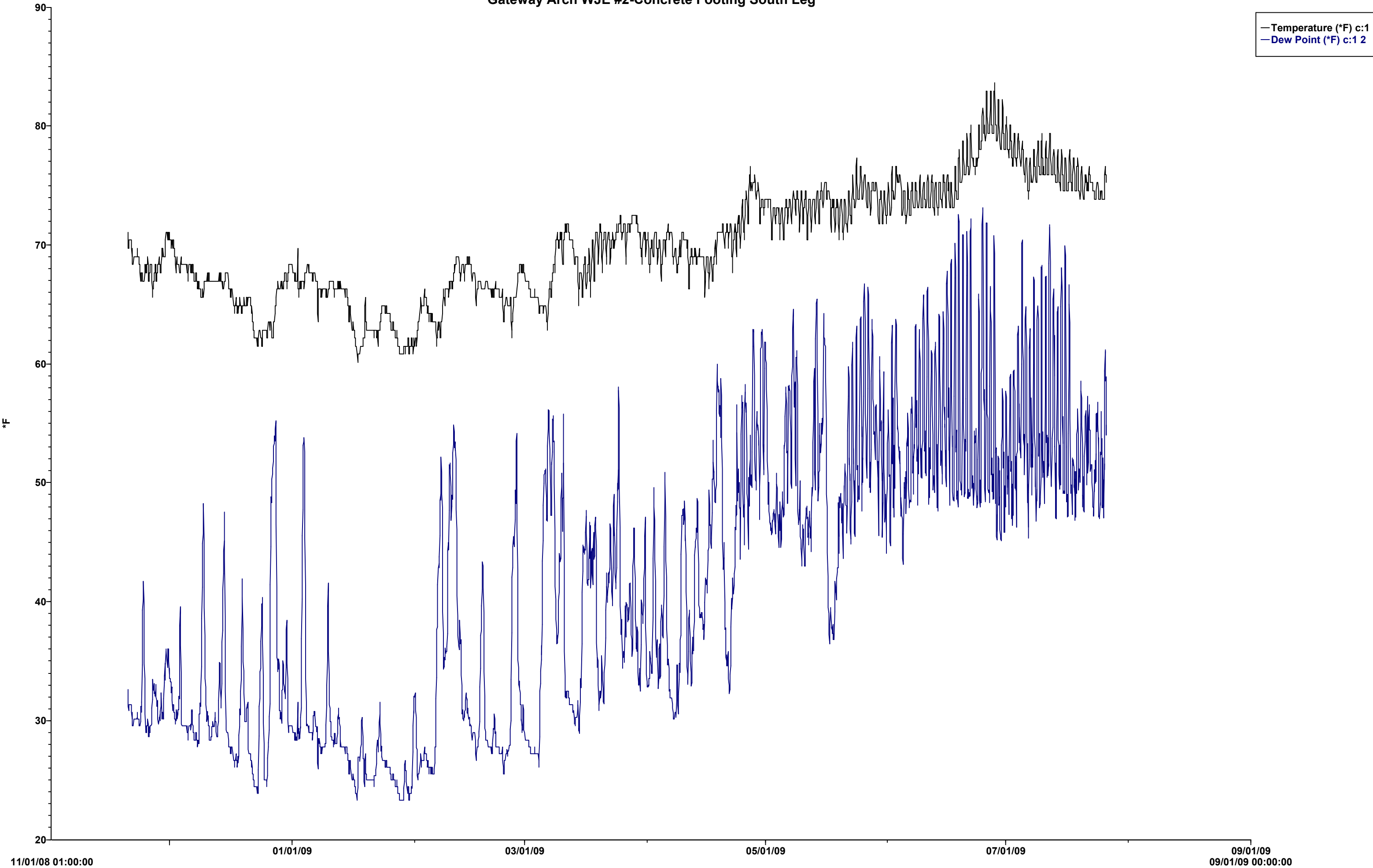
Table 1. Interior Temporary and Humidity Readings, November 12, 2008

Location (Listed from bottom to top)	Dry Bulb Temp, °F	Dew Point, °F	RH, %
Central Museum	72	32	23
North Exhibit Space	70.5	28	20
North Base	69	23.2	18
Station 70	64.5	21.8	19
Station 65	61	22.5	22.4
Station 61	65.6	20.8	17.1
Station 57	62	21.8	21
Station 53	58.4	21.8	24
Station 49	60.8	20.2	20
Station 44	63.1	19.8	18.5
Station 39	66.3	21.4	17.9
Station 34	65	20.9	18.1
Station 29	66	20.9	17.6
Station 24	65.3	22	19
Station 19	65.3	18.6	21.4
Station 14	67.4	24.1	19
Tram load zone at top	70	26.6	18.4
Observation area at top	74	21.2	14
Exterior at top	43.5	12.6	28

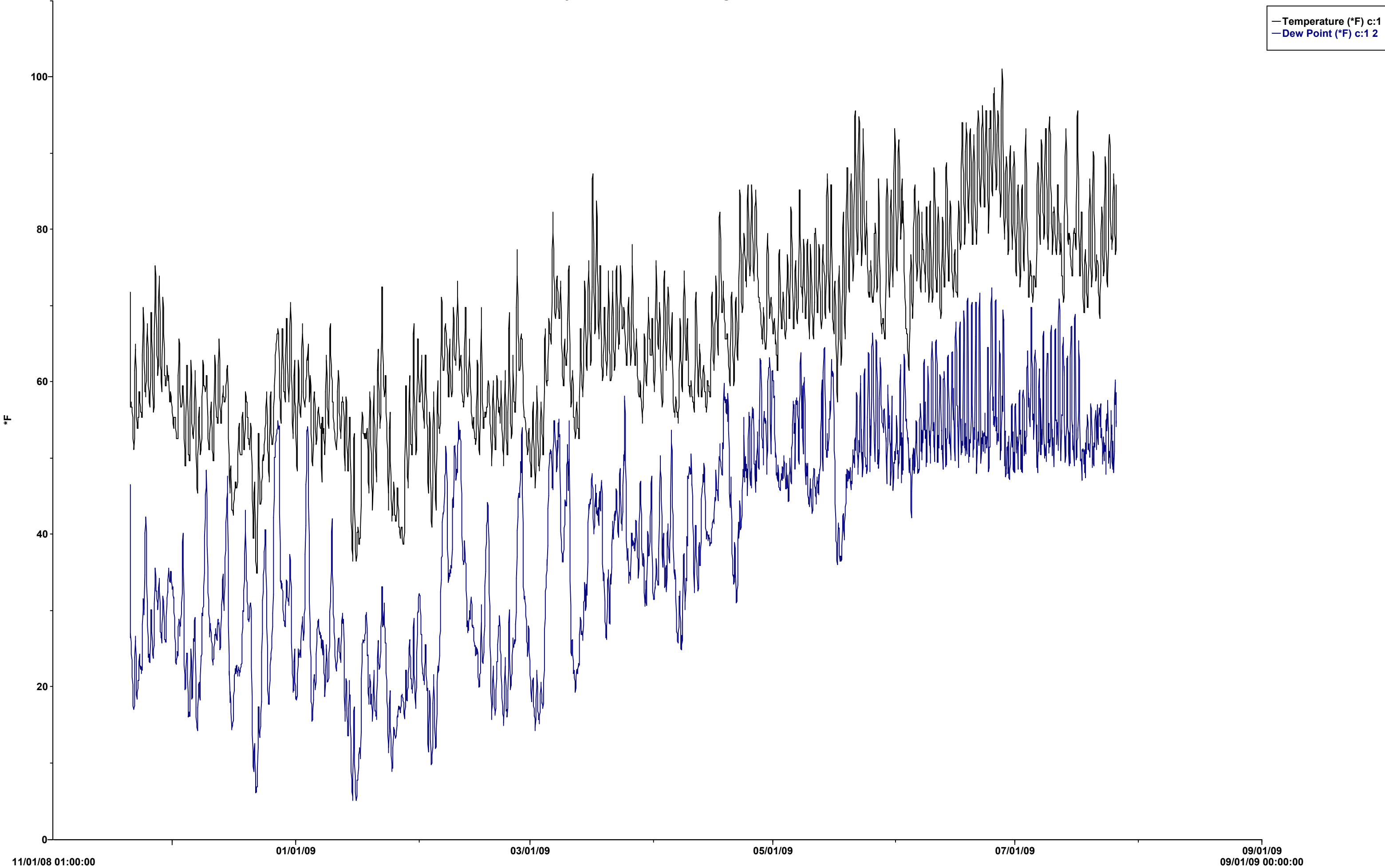
Gateway Arch WJE #1-Exterior



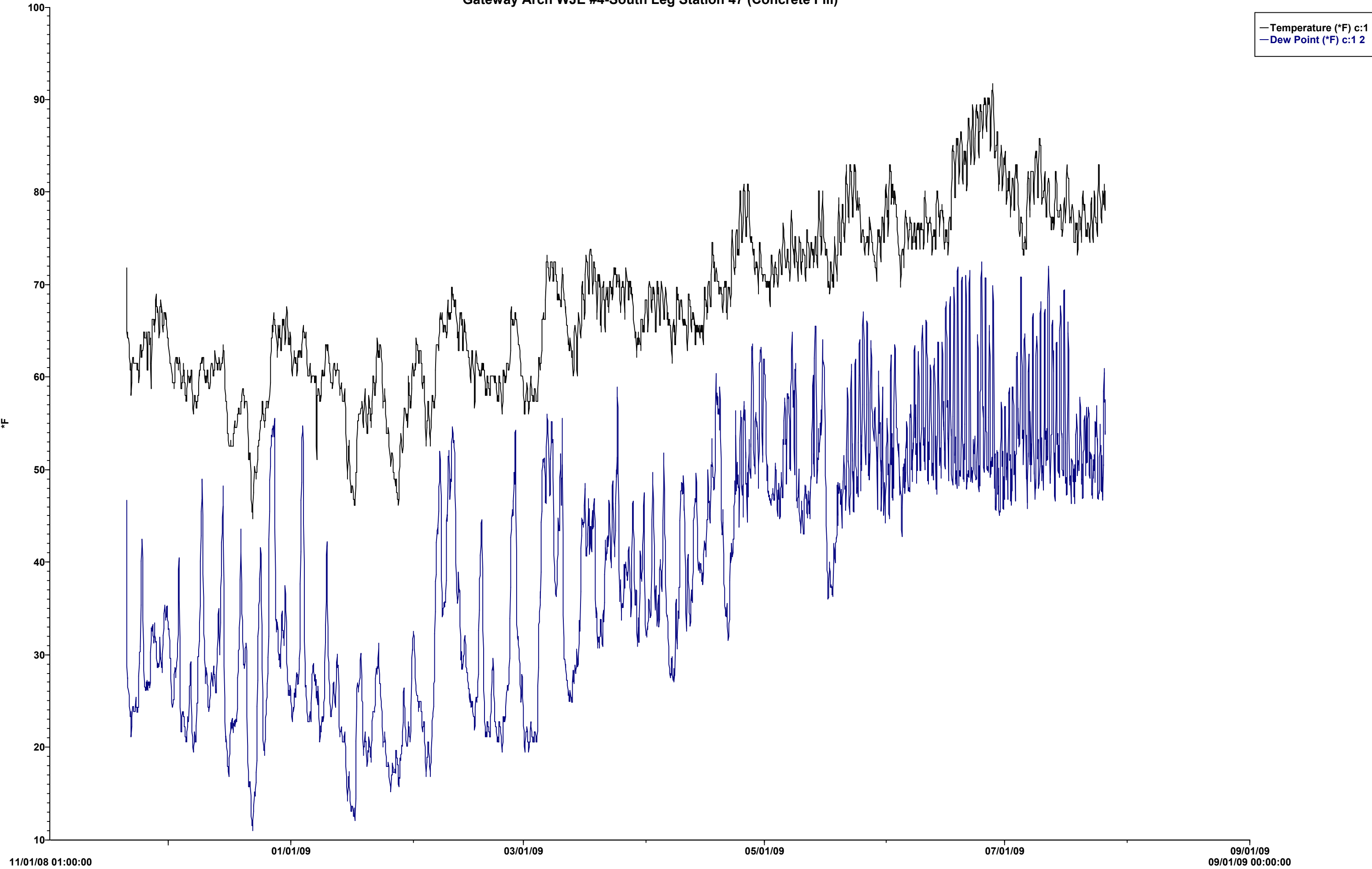
Gateway Arch WJE #2-Concrete Footing South Leg



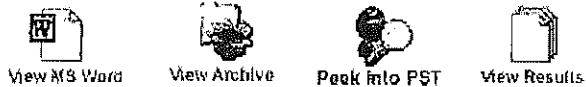
Gateway Arch WJE #3-South Leg Station 14



Gateway Arch WJE #4-South Leg Station 47 (Concrete Fill)



**Appendix E – Install New Entrance Ramps to Arch Monument and Visitor Center,
PMIS 150546**



< p m i s >

Log Out

Create Profile

Search

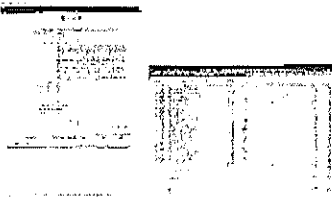
Report

Help

Project Identification - PMIS 150546	
Project Title: Install New Entrance Ramps To Arch Monument and Visitor Center	Project Total Cost: \$9,100,718.35
Park/Unit: Jefferson National Expansion Memorial	Region: Midwest
States: MO	Congressional Districts: MO01
Related Parent FMSS Work Order Number: 2706663	Reference Number:
Project Type: Facility , Historic , Imported from PST	Financial System Package Number: JEFF 150546
Contact Person: Pete Swisher	Contact Phone: 314-655-1623

Project Status - PMIS 150546	
Date Imported: 01/16/09	Review Status: WASO-Reviewed on 05/05/2009
Date of Last Update: 03/30/09	Updated By: Kathy Schneider, Kathleen_Willcuts@Nps.Gov, Kathleen Willcuts, Keschneider@Nps.Gov

Project Narratives - PMIS 150546
Description <p>With the event of 9/11, security and anti-terrorism efforts in public facilities have come under strict scrutinization regarding physical threats. As the Arch National Monument has been deemed an Icon Park, we have had numerous inspections from the Dept. of State, State Safety and Health, and the Dept. of Interior we have been required to change the Arch entrance access to provide protection to the Arch Monument and Visitor Center in the event of an explosive attack. This will encompass reconfiguring the entrances from a different location and providing for "switch back" hallways to reduce the impact of an explosion. And finally building up exterior walls to the monument as well as the adjacent railroad tunnels will be included in the design and construction of the new access points.</p> <p>This construction will also provide for ADA within the scope of design. 2704831 Construct New Entrance Ramps</p> <p>Formerly PMIS Project 9543</p>
Justifications



With the event of September 11, security and anti-terrorism efforts in public facilities have come under strict scrutinization with regard to their ability to repel physical threats. The Gateway Arch at Jefferson National Expansion Memorial is an iconic figure of international stature, and as such has had numerous inspections from the Department of State, State Safety and Health offices, and the Department of Interior. Through these inspections, potential changes to the entrance stations have been identified to provide protection to the Arch and its interior visitor center in the event of an explosive attack. This change will encompass reconfiguring the entrances from a different location and providing for "switch back" hallways to reduce the impact of an explosion. Additionally, building up exterior walls to the monument as well as the adjacent railroad tunnels will be included in the design and construction of the new access points. This construction will also provide for ADA accessibility within the scope of design. Construction of new ADA accessible entrance ramps into the Arch underground visitor center complex will bring the park into compliance with all ADA guidelines / laws and make the Arch accessible to all visitors. Since the Arch was constructed in the early 1960's, visitation has more than doubled. Visitation now exceeds 4 million annually. Current demographic projections predict the average age of the general population as getting higher. Based upon better health care than that provided a half century ago, life expectancy for the average U.S. citizen has been extended significantly. As a result, the population includes substantially more persons with physical disabilities and mobility impairments. The park receives 30 +/- complaints annually for not being readily accessible for the mobility impaired populations; frequently these complaints are also made public through local media outlets. Funding this project would provide a much needed service for those with disabilities and mobility impairments. It is not unusual to have daily visitation of over 30,000 during peak summer months.

Measurable Results

Completing this project will bring the park into compliance with directives from the Dept. of State, Dept. of Interior, State Safety and Health; and current emergency evacuation and anti-terrorism protection. This new construction will also provide for the identified ADA accessibility compliance.

Project DOI Categories of Facilities Maintenance and Construction Needs - PMIS 150546

Total Project Score/Ranking (imported from PST) — 925		DOI Ranking Factor Score — 900	
Deferred Maintenance Needs		Capital Improvement Needs	
Critical Health and Safety Deferred Maintenance Need	0%	Critical Health and Safety Capital Improvement Need	100%
Critical Resource Protection Deferred Maintenance Need	0%	Critical Resource Protection Capital Improvement Need	0%
Critical Mission Deferred Maintenance Need	0%	Energy Policy, High Performance Sustainable Building Capital Improvement Need	0%
Other Deferred Maintenance Need	0%	Code Compliance Capital Improvement Need	0%

	Other Capital Improvement Need	0%
--	--------------------------------	----

Project Activities, Assets, Resources and Emphasis Areas and GPRA Goals - PMIS 150546	
Activities <ul style="list-style-type: none"> • Construction • Compliance • Hazard/Pest Management • Protection 	Assets <ul style="list-style-type: none"> • Building
Emphasis Areas <ul style="list-style-type: none"> • Accessibility Compliance • Security Enhancement • Deferred Maintenance • Health and Safety 	GPRA Goals and Percent Values <ul style="list-style-type: none"> • Visitor Safety(accidents and fatalities), 90% • Visitor satisfaction overall, 10%

Project Prioritization Information - PMIS 150546	
Unit Priority: 40 IN FY 2009	Unit Priority Band: HIGH

Project Capital Asset Accounting Determination Results Summary - PMIS 150546
--

Capital Asset Accounting Determination (CAAD) has been done, and all accounting beginning with pre-design planning must be in accordance with the following rules:



Arch
Visitor
Center

Account coded as "Heritage Assets" is REQUIRED to track these assets separately from other assets and costs, beginning with Pre-Design. Heritage Asset Accounts are established by coding "HP" in the G/L Post Type field and "HA" in the PROJ GROUP field of the FFS PROJ table when they are established.

Project Assistance Needs - PMIS 150546
Is Assistance Needed: No

Related OFS Funding Requests - PMIS 150546
Request ID: 4804 Request title: <u>Maintain and Repair Aging Park Physical Facilities</u>
Request ID: 11926 Request title: <u>Provide Additional Law Enforcement and Security</u>
Request ID: 10973 Request title: <u>Provide Law Enforcement & Security</u>

Project Funding Component - PMIS 150546A
--

Funding Component Title: Install New Entrance Ramps To Arch Monument and Visitor Center		Funding Component Request Amount: \$9,100,718.35			
Funding Component Reference Number (Multi-purpose):		Funding Component Type: Non-recurring			
Funding Component Description: This component represents work covered by child work orders of FMSS Parent WO 2706663 with target start dates in FY 2009.					
Initial Planned FY: 2009		Requested Funding FY: 2012			
Review Status: Region-reviewed on 03/30/2009		Funded Amount:			
Date of Park Submission: 02/12/2009		Submitted By:			
Upper-level Review Status:		Fee-demo Submission Number:			
Formulated FY: 2009		Funded FY:			
Formulated Program: 5 Year Plan		Funded PWE Accounts:			
Formulated Funding Source: Line Item Construction		Funded Funding Source:			
Related PEPC Information					
Related PEPC Project Number		Compliance Status			
		Expected Compliance Date			
No Related PEPC Project Numbers Specified.					
Component Cost Estimates					
Labor Cost Type: Contract					
Estimated By: Imported From PST		Date of Estimate: 01/16/2009			
Estimate in 2009 dollars		Class of Estimate: C			
Item	Description	Qty	Unit	Unit Cost	Item Cost
Estimated Labor Cost	Imported from PST	1	Lump	\$2,483,133.92	\$2,483,133.92
Estimated Material Cost	Imported from PST	1	Lump	\$3,812,972.36	\$3,812,972.36
Estimated Equipment/Tool Cost	Imported from PST	1	Lump	\$204,406.82	\$204,406.82
Estimated Service Cost	Imported from PST	1	Lump	\$0.00	\$0.00
Component Net Construction					\$6,500,513.10
Escalation Adjustments					
Item	Description				Item Cost

Escalation 4% per year		\$0.00							
Component Net Construction (Escalated to FY 2009)		\$6,500,513.10							
Second-level Cost Add-ons									
Add-on	Description	Amount							
Compliance Default(%): 5% Default(\$): \$325,025.66	Updated as (5%) of Project Net Construction after being imported from PST.	\$325,025.66							
Pre-design Default(%): 5% Default(\$): \$325,025.66	Updated as (5%) of Project Net Construction after being imported from PST.	\$325,025.66							
Final Design Default(%): 10% Default(\$): \$650,051.31	Updated as (10%) of Project Net Construction after being imported from PST.	\$650,051.31							
Supplemental Services Default(%): 2% Default(\$): \$130,010.26	Updated as (2%) of Project Net Construction after being imported from PST.	\$130,010.26							
Construction Management Default(%): 8% Default(\$): \$520,041.05	Updated as (8%) of Component Net Construction after being imported from PST.	\$520,041.05							
Construction Contingency Default(%): 10% Default(\$): \$650,051.31	Updated as (10%) of Component Net Construction after being imported from PST.	\$650,051.31							
Component Funding Request (Component Net Construction Escalated to FY 2009 + Total Add-on Amount)		\$9,100,718.35							
Eligible Funding Categories and Funding Priorities									
Funding Category	Unit Priority at Formulation	Priorities by Eligible Funding Sources	Year Unit-Prioritized						
Line Item Construction	40	<table border="1"> <tr> <td>Funding Source</td> <td>Regional Priority</td> <td>National Priority</td> </tr> <tr> <td>Line Item Construction</td> <td></td> <td></td> </tr> </table>	Funding Source	Regional Priority	National Priority	Line Item Construction			2009
Funding Source	Regional Priority	National Priority							
Line Item Construction									
Line Item Construction CBA Factors - PMIS 150546A [Total Importance of Advantages Score: 0] [Advantage to Cost Ratio: 0]									
<p>API value [1 - 100]: 100</p> <p>Current Average FCI value of all assets: 0.168</p>									

Projected Average FCI value of all assets: 0.108

Current Annual Operating Cost: 443676

Projected Annual Operating Cost: 465860

Factor: Provide Safe Visits and Working Conditions
Importance of Advantage Score: 0

Description of Current Conditions/Deficiencies

1. What basic facilities and services (such as comfort stations, shelter, orientation/safety information, and safe access) are currently available in the park and/or sub-area affected by the project?

The current entrance ramps have been narrowed to accommodate the security checkpoints, including magnetometers, x-ray equipment, three to four Guards and one Law Enforcement Ranger to screen visitors as they enter the underground complex.

The current operation is not in compliance with recommendations from the DOI Office of Law Enforcement Emergency Services, Department of State and two site assessments by 3rd party security specialist.

2. What is the existing situation with respect to public health, safety, and welfare, especially for park visitors? How many visitors or other members of the public are affected by the existing situation? What would be the result for park visitors and other members of the public if this project were not completed?

Potentially, all 4.2 million annual visitors to the park are exposed to issue. If this project were not completed, there is a threat to the security of the main structures of the Arch Visitor Center and Arch Monument, and the potential for a blast impact that could damage the structural integrity of the Arch Monument and decimate the visitor center causing injuries, loss of life to occupants within the Arch Monument, Visitor Center and potentially those on the grounds, adjacent railroad lines, city streets, and highway systems.

Those who visit the park who are mobility-impaired or visit with mobility-impaired family members (parents, siblings, or children) currently cannot be fully accommodated for access. This project will provide access to this population as well.

3. What are the specific risks to the public health and/or safety? What is the probability, immediacy, and/or timeframe associated with these risks? What would result if the risk were not eliminated? How serious and extensive would the effects be?

In the event of a bomb blast there could be flying debris, structural damage to the Arch monument causing a threat to those within the building, as well as, on the grounds and potentially the neighboring railroad tunnels, and public roads and highways that run through and adjacent to the park site.

With the changes of the entrance to encompass walled ramps with switchbacks there would be a reduction in the blast impact in the event of an explosion. This would reduce the damage to the visitor center and help to maintain the integrity of the Arch Monument structure. This project would also reduce the blast impact to the adjacent railroad tunnels by providing impact reducing materials within those adjacent walls and buffer zones.

4. What basic administrative facilities (such as restrooms, shelter, efficient workspace, and

safe access) are currently available in the park and/or sub-area affected by the project?

N/A

5. What is the existing situation with respect to employee health, safety, and welfare? How many employees are affected by the existing situation? What would be the result for them if this project were not funded?

If this project is not funded, it will have the same impact on employees as on visitors, e.g., in the event of a terrorist attack there would be massive loss of life, injuries, and for ADA some will not be able to be accommodated.

6. What are the specific risks to employee health and/or safety? What are the probability, immediacy, and/or timeframe associated with these risks? What would result if the risks were not eliminated? How serious and extensive would the effects be?

The same as for the park visitors. With the current world economic and social climate this threat is very real. As an icon park the risk is evaluated as highly probable. In the event of an explosion there would be catastrophic injuries and loss of life impacting locally, regionally, and nationally the confidence of people for safety in their daily lives.

7. Upon what information or authority have these predictions been made?

The Arch Monument has been deemed an Icon Park, we have had numerous inspections from the Dept. of State, State Safety and Health, and the Dept. of Interior with recommendations for a change in the entrance access of the Gateway Arch Monument and Visitor Center.

8. What citations, court orders or other legal direction has the park received based on violation of regulations, codes or other legal standards of health, safety?

After numerous inspections from the Dept. of State, State Safety and Health, and the Dept. of Interior we have been required to change the Arch entrance access to provide protection to the Arch Monument and Visitor Center in the event of an explosive attack.

Project advantages in protecting public health, safety, and welfare

9. How will the proposed project provide basic visitor facilities and services and/or allow the park to meet established standards of health, safety, and welfare? How many visitors or other members of the public would be effected?

Rerouting the entrances will provide switchbacks which will lessen the effects of a significant blast and reduce the number of potential victims in the event of an explosion. The design of the switchbacks will also provide ADA access to the site, and would be in compliance with the Architectural Barriers Act, Section 504 and Rehabilitation Act of 1968. This site currently provides services for three million visitors annually.

10. What alternatives have been considered to address these issues without construction (such as closing a given park area), outside the park, or through a non-NPS source (such as another public agency or commercial facility)?

There are no other viable alternatives for this issue. The current construction of the Arch entrances provide no other realistic avenue of approach. When the Title I and II services were initiated in the mid-1990's, consideration was given to elevators, moving conveyers, etc. None of these were acceptable as the park grounds are an approved Cultural Landscape on the national register, and the introduction of a "built" environment onto the landscape did not make for a realistic alternative. The Arch Monument is the primary visitor attraction of the park with high local, regional, and national significance.

Project advantages in protecting employee health, safety, and welfare

11. How will the proposed project provide basic administrative facilities and/or allow the park to meet established standards of health, safety, and welfare? How many employees would be effected?

This project will ensure the security of the main structures of the Arch Visitor Center and Arch Monument, and reduce the potential for a blast impact that could damage the structural integrity of the Arch Monument and decimate the visitor center. This would affect 110 employees assigned to this work site and an additional 50 employees who work on the grounds and provide administrative oversite to the park with business in the buildings on a daily basis. This project would also provide compliance with the Architectural Barriers Act, Section 504 and the Rehabilitation Act of 1968.

12. What alternatives have been considered to provide comparable facilities and services without construction, outside the park, or through a non-NPS source (such as rental housing or another public agency or commercial facility)?

Not feasible.

Factor: Protect Natural and Cultural Resources
Importance of Advantage Score: 0

Description of Resources

13. What is (are) the nature, extent, quantity, and complexity of the resource(s) (e.g., specific species, watershed, ecosystem, archeological resources, cultural landscape, historic structures, museum objects, ethnographic resources, etc.)?

This project directly affects the Jefferson National Expansion Memorial, the 630' tall stainless steel Gateway Arch. Located on 90+ acres in downtown St. Louis, Missouri, the park is a National Historic Landmark designation site and is on the National Register. The Arch is on the List of Classified Structures (LSC) as Structure Number HS-4 and IDLCS number 00735. Legal mandates to preserve and interpret the Gateway Arch include Executive Order 7523 (December 21, 1935), the Act of May 17, 1954 (68 Stat. 98), the Historic Sites Act of 1935, and the National Historic Preservation Act of 1966.

14. What is the significance (local, state, regional, national) of the resource(s), including any special designation(s) (e.g., wilderness, World Heritage site, National Natural Landmark, Biosphere Reserve, federally listed threatened or endangered species, National Historic Landmark, listed on National Register of Historic Places, etc.)?

Jefferson National Expansion Memorial was designated in 1935 to commemorate significant events in the history of the United States, including the Louisiana Purchase, the Lewis and Clark Expedition, the westward movement of American explorers and settlers, the establishment of the first cathedral and civil government west of the Mississippi River, and the debate over slavery raised by the famous Dred Scott case. The Gateway Arch symbolizes St. Louis' role as the "Gateway to the West." It is a National Historic Landmark. The park has a National Historic Landmark designation and is on the National Register. The Arch is on the List of Classified Structures (LCS) as Structure No. HS-4 and IDLCS no. 00735.

15. How is (are) the resource(s) comparable to others in the region or National Park System either ecologically or in cultural associations?

There are no other cultural resources of comparable design in the world. The 630' high stainless steel Gateway Arch is an internationally recognized symbol. It is as recognized nationally and internationally as the Statue of Liberty, the Washington Monument, the White House, Independence Hall, and the Eiffel Tower. Built in the early 1960's to last 1000 years,

it is the nation's tallest national monument.

16. What policy or legal mandates or park goals for resources management are related to the resource(s)?

Historic Sites Act of 1935 -Executive Order 7523 of December 21, 1935 -Act of May 17, 1964 (68 Stat. 98) -National Historic Preservation Act of 1966

Project advantages in preventing the loss of resources (e.g., stabilization)

17. What is the specific threat to the resource(s)?

In the event of a bomb blast there would be potential for damage to the structural integrity of the Arch Monument and decimate the visitor center

18. What will result if the threat is not eliminated?

Potential for injuries, loss of life to occupants within the Arch Monument, Visitor Center and potentially those on the grounds, adjacent railroad lines, city streets, and highway systems in the event of an explosive event.

19. What is the immediacy or timeframe of the threat?

With the current world socioeconomic status and the icon status of the Arch Monument, the focus of extraordinary security measures, the Gateway Arch Monument has the possibility of threat from terroristic activities.

20. What is the probability that the resource(s) will be lost?

In the event of a blast there would be damage the structural integrity of the Arch Monument and massive damage to the visitor center. The structure could be rebuilt, but in today's costs and OSHA requirements the cost of the Arch Monument would be around \$520,122,065.00 and the cost of the Visitor Center would be \$31,211,796.00.

21. Upon what information or authority have these predictions been made?

After inspections from the Dept. of State, State Safety and Health, and the Dept. of Interior we have been required to change the Arch entrance access to provide protection to the Arch Monument and Visitor Center in the event of an explosive attack. The park also had a blast assessment study with recommendations of alternatives to reduce injury, loss of life, and limit damage to the structural integrity of the Gateway Arch Monument and Visitor Center.

22. How will the proposed project reduce or eliminate the threat?

This project will move the current chute style ramp to an area where visitors will be further from the main structure for the security check in, thus providing security checks to remove threats further from the main structure. The current design would funnel a blast straight into the lobby of the visitor center providing massive damage. Moving the entrance and installing multiple switchbacks would reduce the blast impact and damage to the visitor center.

Project Advantages in maintaining or improving the condition of resources

23. What is the current condition of the resource(s)?

N/A

24 How will the proposed project affect the condition of the resource(s) (e.g., species or ecosystem restoration, disturbed land restoration and revegetation, preservation of an archeological resource, rehabilitation or restoration of a historic structure, or conservation of a museum object -- including preventative conservation provided by a museum collection

storage facility)?

N/A

Factor: Improve Visitor Enjoyment Through Better Service and Educational and Recreational Opportunities

Importance of Advantage Score: 0

Description of Current Visitor Experience

25. What is (are) the nature, extent, and complexity of the current visitor (e.g., park and/or subarea visitation -- annual total as well as average peak-season day, type and nature of access to park and/or subarea, available park facilities and services, available educational and recreational opportunities, type and nature of visitor activities, availability of alternative facilities and services outside the park, etc.)?

There were over three million visitors to the park in 2007. Visitors to the Arch include both individuals and organized groups such as school groups, scout groups, and tour groups. Visitors include local area residents and international tourists from across the nation and around the world. Year-around interpretive programs and museum exhibits are supplemented by numerous special activities throughout the year, including Black History month programs, women's history month programs, the annual storytelling festival, etc. Visitation is as high as 30,000 per day during peak periods. During large special events visitation has surpassed 500,000 per day. This project will enable ALL visitors to enter/exit the two main entrances to the Arch underground visitor center complex. The significance of the visitor experience includes visiting the Arch and underground visitor center complex which houses two theaters (one of which is an IMAX type and the only one of its kind in the National Park Service), a tram transportation system to carry visitors to the top of the 630' high Arch, and the 43,000 square foot Museum of Westward Expansion. Ongoing Ranger-led interpretive programs, curriculum-based school programs, self guided museum tours, movie viewing, and tram rides to the top of the Arch are some examples of the ongoing visitor service activities offered on a daily basis in the underground complex. Visitors unable to enter the facility due to mobility or other disabling impairments will be deprived of that experience.

26. What is the current situation regarding visitor facilities (e.g. condition and functional adequacy, current use vs. capacity, long-term sustainability of use, etc.)?

Most visitor facilities are in good to excellent condition. During extended visitation hours in the summer months, visitation to the park runs as high as 30,000 per day. Visitors must often wait in line for hours to purchase tickets for (one or both) theaters or tickets for a tram ride to the top of the Arch. It is not unusual to be "sold out" by noon on such days. In other words, the capacity for tram rides to the top of the Arch is reached regularly during summer visitation periods. Not all visitors can be accommodated due to the slope of the current ramps and are sometimes forced to "wait" outside for the remainder of their families while they visit and experience the underground visitor center complex.

27. What is the current situation regarding visitor experience(s) of the park and/or subarea affected by the project (e.g., available services and opportunities vs. park goals, visitor satisfaction with services and opportunities, etc.)?

Of the visitors able to enter and exit the Arch underground visitor center complex, the satisfaction and understanding rates, as measured through the park's annual visitor satisfaction survey for GPRA goals, is quite high (usually in the 90 percentile). Obviously, those unable to enter the facility are much less satisfied.

28. What is the significance of the visitor experience? How does it compare to others in the region or national park service?

There is only one Arch of this stature in the world. The tram transportation system that carries the visitor to the top of the 630' high Arch structure is a unique, one-of-a-kind system. It requires continuous and specialized maintenance. The ride to the top takes 5 minutes and the ride down about 3 and 1/2 minutes. The Arch underground visitor complex also contains two theaters, one of which is the only National Park Service-owned and operated Giant-Screen Theater (analogous with IMAX). The 43,000 square foot Museum of Westward Expansion was the first American Museum Association-accredited museum in the Service and houses thousands of artifacts. One major, premier exhibit is the Indian Peace Medal Exhibit. There is none other like it in the world. This exhibit contains the largest known collection of Indian Peace Medals in the world, some of them priceless. The exhibit uses animatronic figures (the only ones in existence in the National Park Service) to tell the story of native Americans, the medals themselves, and the role these medals played in early history. There is no comparable experience like this in the Region or the National Park Service.

29. How is visitor use expected to change without the project (e.g., projected visitation, new use trends or activities, etc.)? Upon what information or authority have these predictions been made?

Visitor use eventually decline, or at best remain status quo with or without the project. Not ALL visitors will be able to be accommodated, only those without disabling impairments. This is based not only upon staff observations during days of heavy visitation, but the annual visitor survey conducted as part of the park's GPRA and Strategic Planning process. The park is open year around. From Memorial Day through Labor Day, the park changes its public visitation hours from the normal 9:00 am to 6:00 pm day and extends hours to 8:00 am to 10:00 pm.

30. What policies, legal mandates, and/or park goals for visitor enjoyment are related to the proposal (e.g., approved plans, agreements with other entities, environmental deficiencies, code violations, regulatory actions, court orders, etc.)?

GPRA goals related to visitor safety and visitor satisfaction. In addition, visitor understanding is also directly affected. If visitors are unable to enter the underground visitor center facility, they cannot understand the significance of the park and its themes; nor can they experience, first-hand, the many exhibits, artifacts, and visitor services offered.

Project advantages in improving visitor services and educational and recreational opportunities

31. How will the proposed project change the condition of facilities and/or the visitor experience(s) of the park and/or subarea -- upon completion and in the future (e.g., the type, quality, and availability of services or educational/recreational opportunities; current and projected visitation -- capacity, use patterns, and activities; deficiencies or visitor satisfaction; access to the park or subarea; services and facilities outside the park; etc.)?

This project would provide secure checkpoints of visitors entering the Arch at a reasonable distance from the structure and limit access to screened individuals limiting impact from explosive devices. The project would also enable ALL visitors regardless of mobility or physical impairment to experience all that is offered in this one-of-a-kind facility. Even though on some days the venues offered are at capacity, the park is not at its full carrying capacity. This project would provide a safer, more enjoyable experience for all park visitors, not just the ones with mobility impairments.

32. How many visitors will be affected by these changes?

N/A

Factor: Improve The Efficiency, Reliability And Sustainability Of Park Operations

Importance of Advantage Score: 0**Description of Current Conditions**

33. What is the nature, extent, and complexity of the current park and/or subarea operation affected by the project (e.g., new area or established park, existing facilities and services, budget and staffing, locational factors such as remoteness or proximity to alternative facilities and services, etc.)?

N/A

34. What is the existing situation for park and/or subarea operations and facilities (e.g., costs, staffing, energy use, functional adequacy, environmental deficiencies, long-term maintainability and/or sustainability of operations, etc.)?

N/A

35. How are park operations expected to change without the project (e.g., new operating methods or practices, projected budget and staffing, etc.)? Upon what information or authority have these predictions been made?

N/A

36. What policies, legal mandates, or park goals for park operations are related to the project (e.g., approved plans, agreements with other entities, environmental deficiencies, code violations, regulatory actions, court orders, etc.)?

N/A

Project advantages in improving operational efficiency, reliability, and sustainability

37. How will the proposed project change park and/or subarea operations and facilities -- upon completion and in the future (e.g., costs, staffing, energy use, the quality and availability of services, environmental effects, maintainability, sustainability, etc.). How much will operational costs and staffing be reduced or increased with the project completed?

N/A

38. What alternatives have been considered to provide comparable facilities and services without construction, outside the park, or through a non-NPS source (such as another public agency or commercial facility)?

N/A

**Factor: Provide Cost-effective, Environmentally Responsible, and otherwise Beneficial Development for the National Park System
Importance of Advantage Score: 0****Other project advantages provided to the National Park System**

39. What other benefits or advantages to the park, the national park system, or other entities, not addressed in the responses above, would result from completion of the proposed project?

N/A

40. How would the project provide continuity with or help obtain maximum benefit from previous line-item construction projects or other capital investments?

N/A

41. How would the project improve long-term institutional capability to accomplish the park or NPS mission?

N/A

42. How would the project demonstrate extraordinary organizational leadership or demonstrate innovative approaches that promote conservation and preservation values within and/or beyond the national park system?

The National Park Service has invested considerable capital expenditure in the Jefferson National Expansion Memorial, e.g., building the Arch, landscaping the grounds, constructing the underground visitor center complex, etc., all within the original design by the internationally renown Architect, Eero Saarinen, and Landscape Architect, Dan Kiley. Construction of ADA accessibility ramps into the underground visitor center complex which do not take away aesthetically from the Arch and cultural landscape and do not impact or intrude visually upon the design, is highly desirable. With the current designed ADA ramps, the National Park Service can show its commitment to and leadership in historic preservation and at the same time show its support for all Americans (and non-Americans) with disabilities. It is very important that this project be completed.

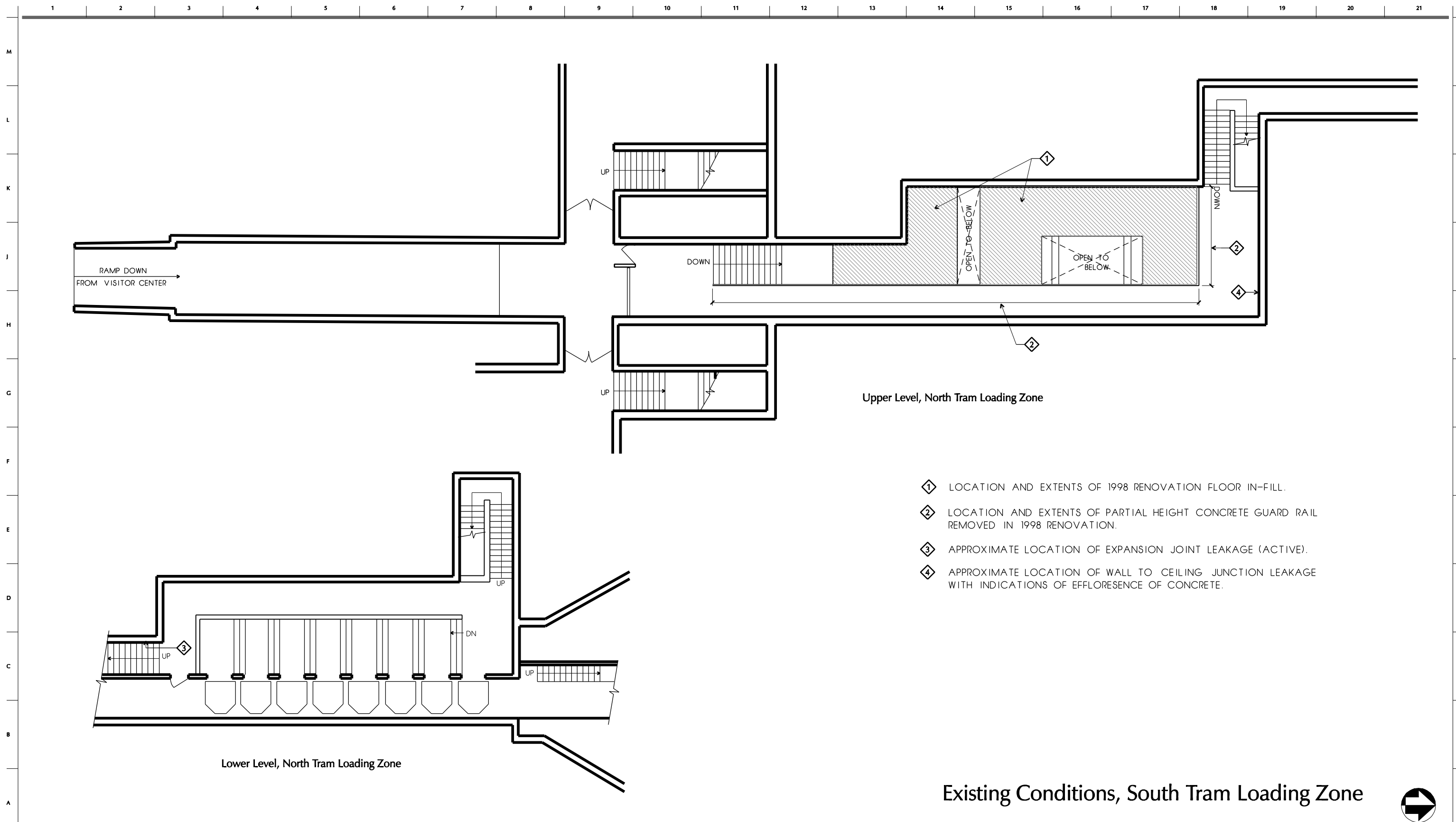
43. How would the project improve park and/or NPS organizational credibility by fulfilling legal mandates, agreements, or other commitments?

It is fairly obvious that the National Park Service could improve its credibility in a number of program areas by fulfilling its responsibilities under the ADA guidelines. Although not mandated by law, the Service could go a long way in showing the disabled community its commitment to and support of these individuals. It's the right thing to do!

44. What benefits or advantages would the project provide to partners, neighbors, communities, or other entities that are not described above?

The City of St. Louis would benefit indirectly and directly. An economic impact statement resulting from a money generation model in 1994-1995 revealed that the Jefferson National Expansion Memorial generates approximately \$48.3 million annually to the St. Louis area from tourism, government expenditures, and park partners; and an additional \$14 million annually is generated in tax revenue. Those figures have at least doubled since that study. Anything that the National Park Service does to attract, retain, enhance, and increase visitors to the St. Louis area, will benefit the entire greater metropolitan community.

**Appendix F – Base Drawings showing Existing Conditions and Conceptual
Smoke Separation Graphics**



- 1 LOCATION AND EXTENTS OF 1998 RENOVATION FLOOR IN-FILL.
- 2 LOCATION AND EXTENTS OF PARTIAL HEIGHT CONCRETE GUARD RAIL REMOVED IN 1998 RENOVATION.
- 3 APPROXIMATE LOCATION OF EXPANSION JOINT LEAKAGE (ACTIVE).
- 4 APPROXIMATE LOCATION OF WALL TO CEILING JUNCTION LEAKAGE WITH INDICATIONS OF EFFLORESCENCE OF CONCRETE.

Existing Conditions, South Tram Loading Zone



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(402) 475-4551

DATE	SHEET	REVISED	SHEET	MARK	APPROVED
					REVIEWED
					REVIEWED
					REVIEWED
					REVIEWED
					REVIEWED



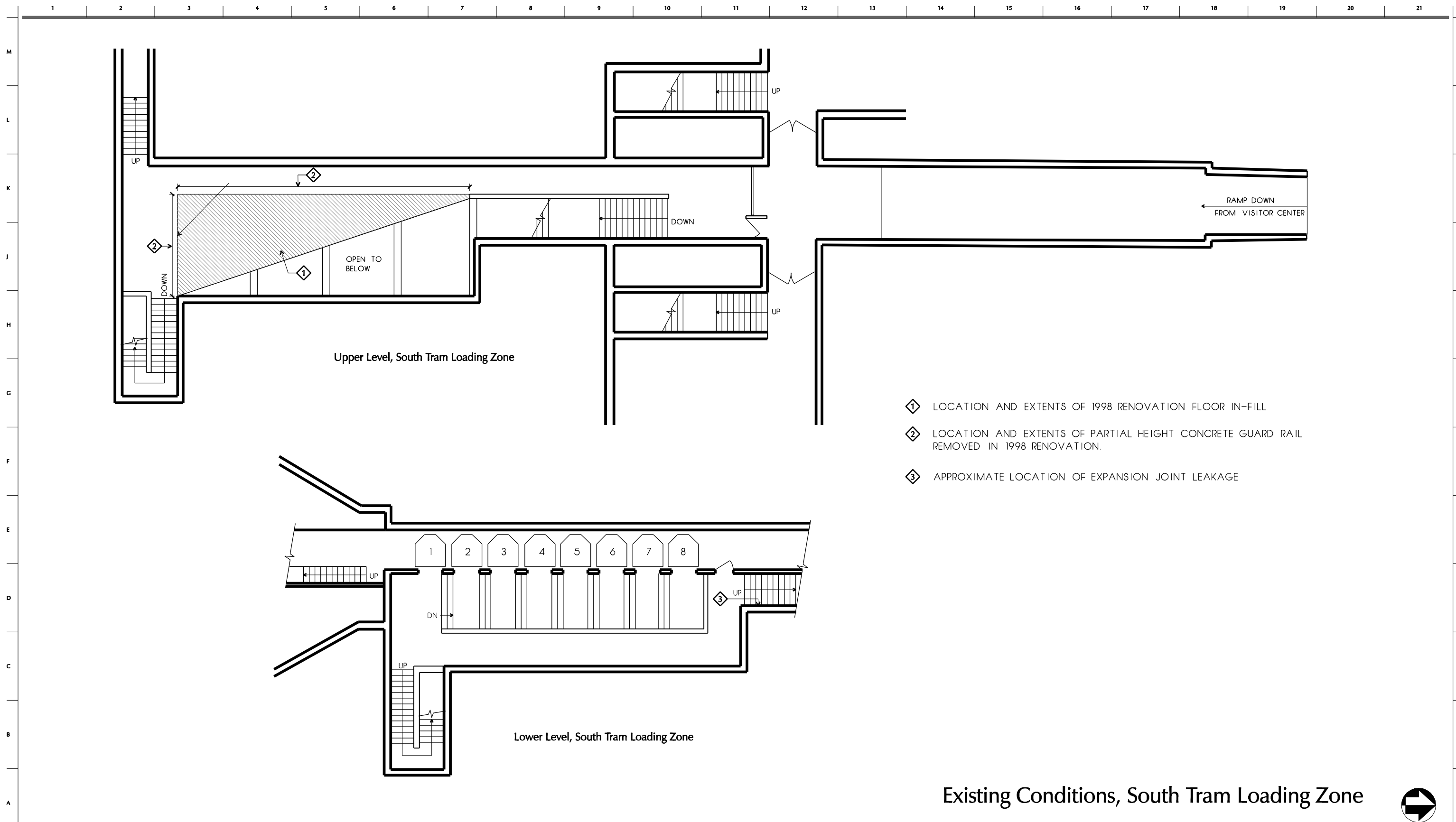
UNITED STATES
DEPARTMENT OF THE INTERIOR

NATIONAL PARK SERVICE
Omaha Regional Center

NPS Jefferson National Expansion
Memorial Gateway Arch

Historic Structures Report

DRAWING NO.	
PKG. NO.	
SHEET	
OF	



- 1 LOCATION AND EXTENTS OF 1998 RENOVATION FLOOR IN-FILL
- 2 LOCATION AND EXTENTS OF PARTIAL HEIGHT CONCRETE GUARD RAIL REMOVED IN 1998 RENOVATION.
- 3 APPROXIMATE LOCATION OF EXPANSION JOINT LEAKAGE

Existing Conditions, South Tram Loading Zone



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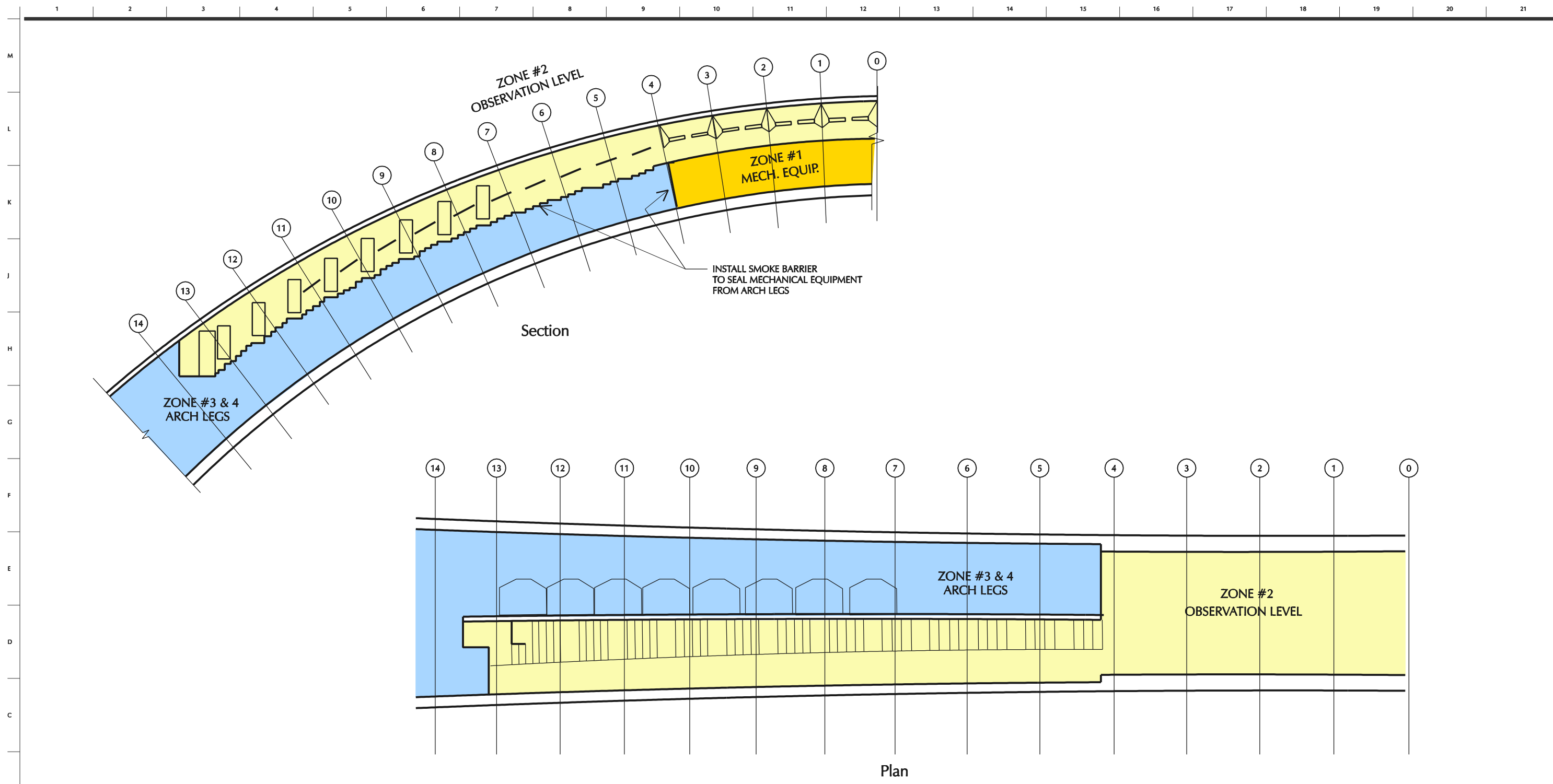
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					REVIEWED	DATE
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					REVIEWED	DATE
					REVIEWED	DATE



UNITED STATES
DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
Omaha Regional Center

NPS Jefferson National Expansion
Memorial Gateway Arch
Historic Structures Report

DRAWING NO.	_____
PKG. NO.	_____
SHEET	_____
OF	_____



Treatment Recommendation Proposed Smoke Separations



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MARK	SHEET	REVISED	SHEET	MARK	APPROVED	DATE
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					REVIEWED	DATE
					REVIEWED	DATE



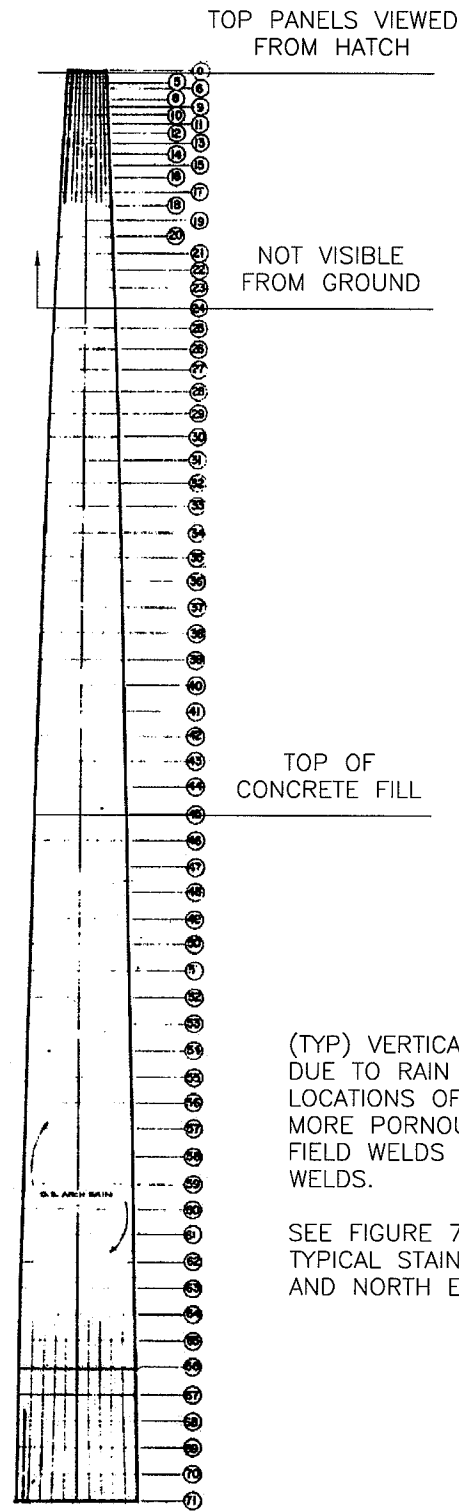
UNITED STATES
DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
Omaha Regional Center

NPS Jefferson National Expansion
Memorial Gateway Arch
Historic Structures Report

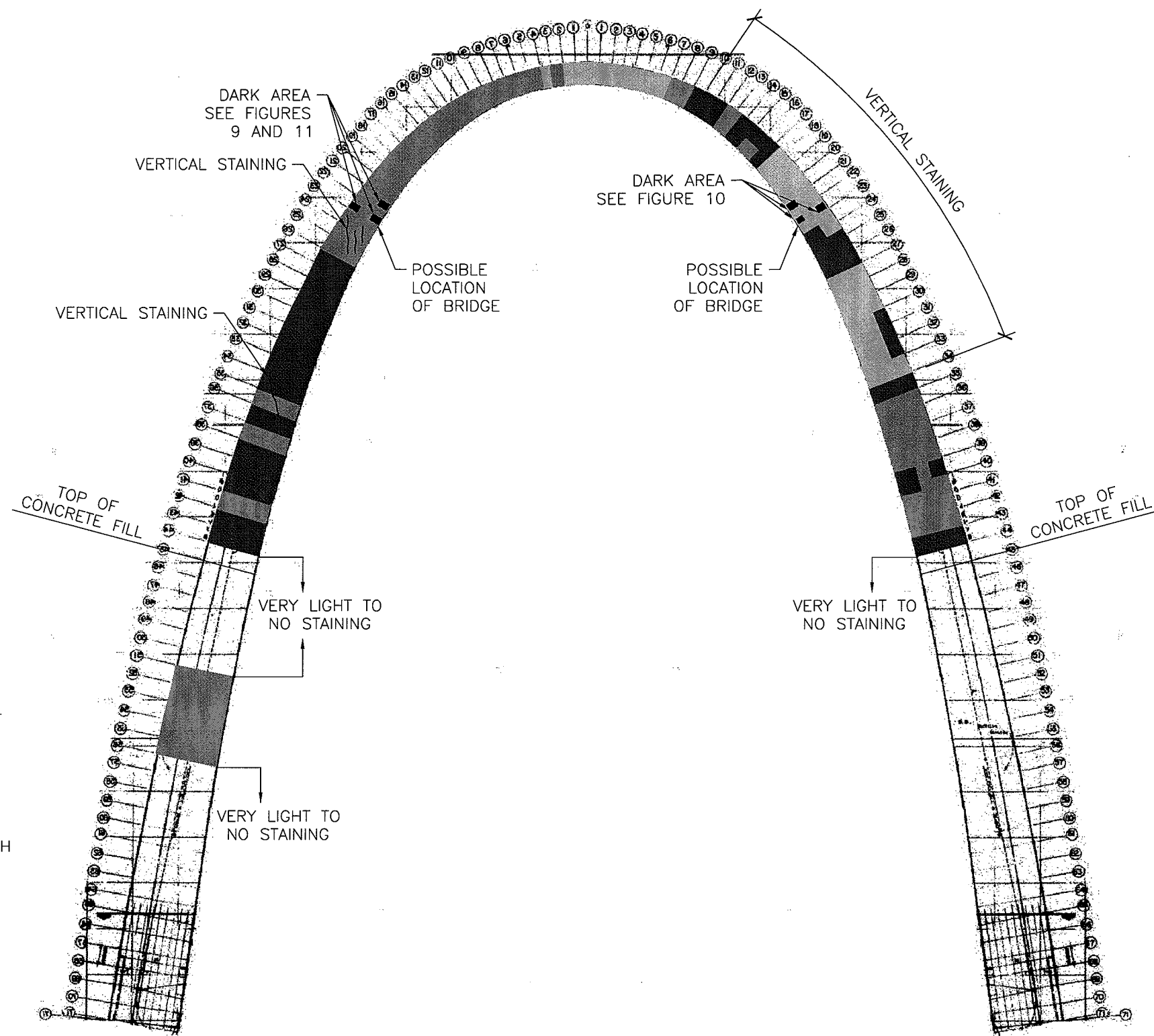
DRAWING NO.	
PKG. NO.	SHEET
OF	OF

Appendix G – Exterior Condition Assessment Drawings

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1 North Elevation
SCALE: N.T.S.



2 East Elevation
SCALE: N.T.S.

LEGEND

- SEVERE STAINING
- MODERATE STAINING
- LIGHT STAINING
- VERY LIGHT STAINING
- DARK AREA
- VERTICAL STAINING

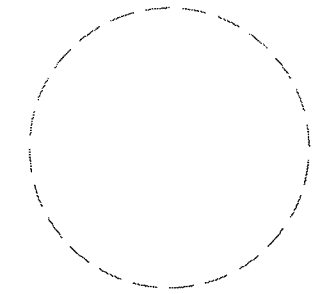
EASILY VISIBLE FROM GROUND
VISIBLE WITH DILIGENT OBSERVATIONS FROM GROUND
VISIBLE WITH BINOCULARS
VISIBLE WITH DILIGENT OBSERVATIONS WITH BINOCULARS
LIMITED AREA OF DARK STAINING OR SCRAPE IN S.S.
AREAS OF STAINING ORIENTATED VERTICALLY

WJE ENGINEERS
ARCHITECTS
MATERIAL SCIENTISTS

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Honolulu | Houston | Los Angeles | Minneapolis | New Haven | New York
Princeton | San Francisco | Seattle | Washington, D.C.

Seal



Consultant

Project

GATEWAY ARCH
Saint Louis, MO

Client

National Park
Service
Omaha, NE

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--	--	--

1 3/27/06 DRAFT REPORT

Mark	Date	Description
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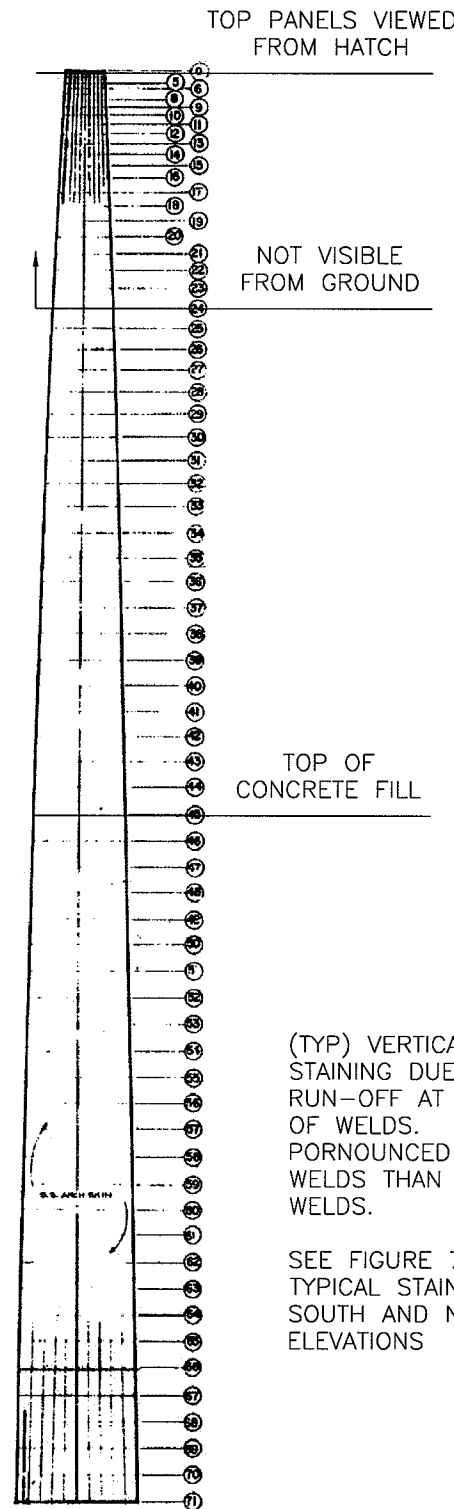
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FIELD CONDITION
SURVEY

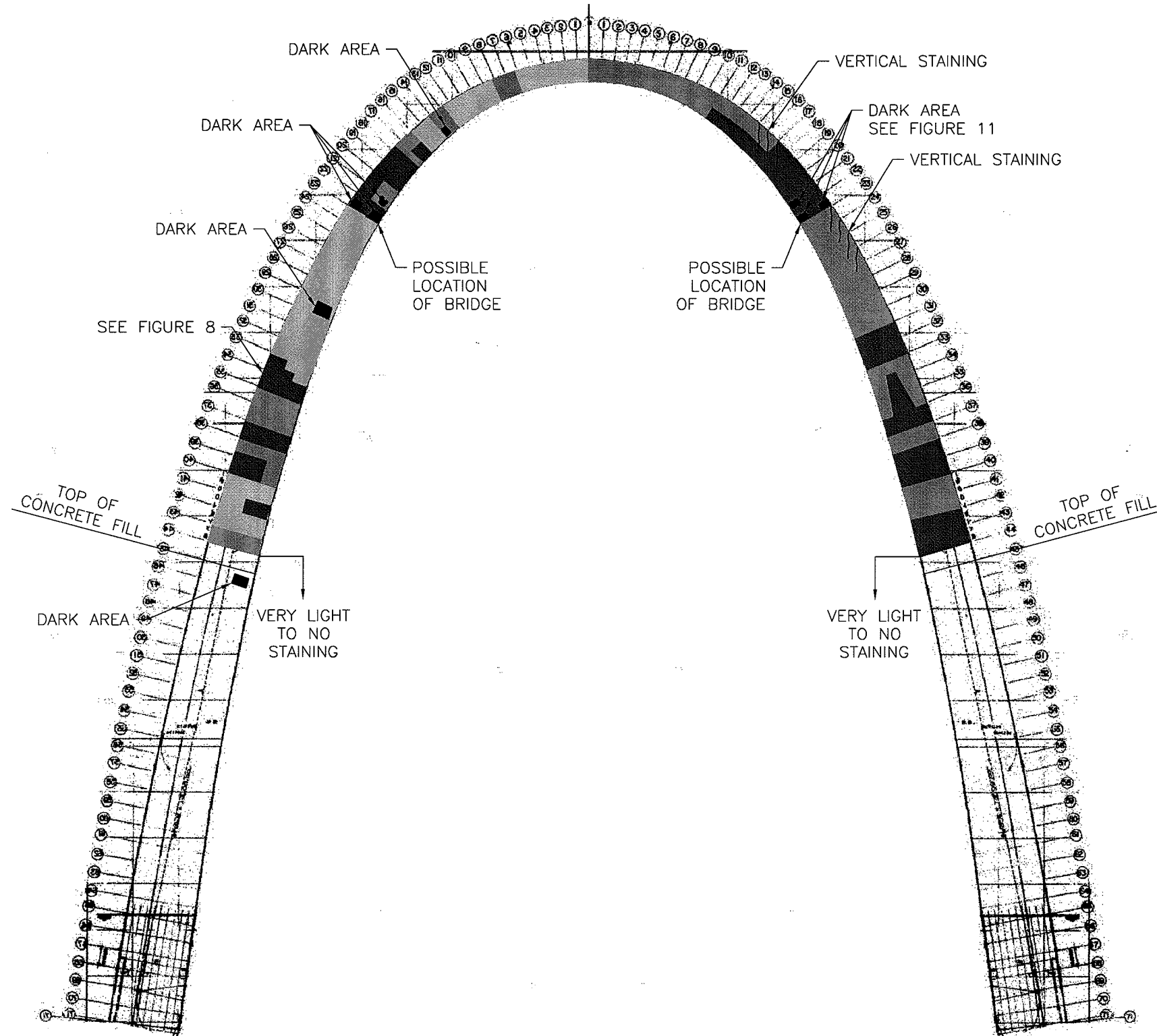
Sheet Title

Sheet No. **SKS-1**

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1 South Elevation
SCALE: N.T.S.



2 West Elevation
SCALE: N.T.S.

LEGEND

- SEVERE STAINING
- MODERATE STAINING
- LIGHT STAINING
- VERY LIGHT STAINING
- DARK AREA
- VERTICAL STAINING

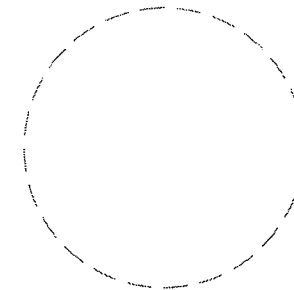
- EASILY VISIBLE FROM GROUND
- VISIBLE WITH DILIGENT OBSERVATIONS FROM GROUND
- VISIBLE WITH BINOCULARS
- VISIBLE WITH DILIGENT OBSERVATIONS WITH BINOCULARS
- LIMITED AREA OF DARK STAINING OR SCRAPE IN S.S.
- AREAS OF STAINING ORIENTATED VERTICALLY

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Seal



Consultant

Project

GATEWAY ARCH
Saint Louis, MO

Client

National Park
Service
Omaha, NE

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1 3/27/06 DRAFT REPORT

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Project No.	2005.3684
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Date	27 MARCH 2006
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Scale	As Shown
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FIELD CONDITION SURVEY

Sheet Title

Sheet No.

SKS-2

Appendix H – Oral History Transcripts

Interview with Ken Kolkmeier

Date: January 14, 2009
Location: Old Court House JNEM Conference Room
Time: 9:30 a.m.
Attending: Ken Kolkmeier, Al O'Bright NPS Historical Architect, Steve Kelley WJE, Bob Moore NPS JNEM, Victoria Dugan NPS JNEM, and Dan Worth BVH Architects
Summary by: Dan Worth and Steve Kelley

Note: the following is a summary of meeting notes taken during an interview with Mr. Ken Kolkmeier on January 14, 2009, at the JNEM. The meeting was held as part of the development of a Historic Structure Report for the Gateway Arch. The intention of the meeting was to have an oral history but Mr. Kolkmeier declined to be taped. As a result, notes were taken during the interview and the following compellation was prepared.

A list of questions had been given to Ken prior to the meeting. Ken mentioned that he had reviewed the questions and made some notes prior to the interview and could follow these in order if desired. He also brought some personal photos and articles from his files.

Bob Moore opened the meeting by stating that this is an interview with Ken Kolkmeier, who worked as project engineer for Pittsburgh-Des Moines Steel on the construction of the Gateway Arch. Bob asked if Mr. Kolkmeier could "tell us the date and place of your birth, where you went to school, and how you came to be employed as an engineer on the Arch project?"

Ken responded that "I was born in St. Charles, Missouri. I was the project manager for PDM on the Arch project. I coordinated the erection procedure with McDonald Construction and fabrication with the procedures for doing the field work." Ken also noted that he was surveyor for the US Army in World War II. Following WWII he worked at PDM working his way up to a Vice President position. From PDM he moved to the Nooter Corporation, a steel erection company in St. Louis, as VP for construction until 1995 when he retired.

Ken noted that while working on the Gateway Arch he worked closely with Dr. Bandel at Severud Engineering and had high respect for him. Dr. Bandel did the design for the Crystal Cathedral as well. Dr. Bandel had commented to Ken that the creeper cranes would be like mosquitoes on the Arch.

Bob Moore asked "Did you visit the grounds of the Arch before the project began? At what stage was the Arch project when you first went to work there?"

Ken responded that "No, I didn't. I was on a project in Arkansas and saw the drawing at one construction meeting. While in Little Rock around 3 p.m. a call from the division manager told me to be in Pittsburgh tomorrow at 9 a.m. I asked him what's going on and he said you know that Arch project? "Well, you're going to be the project manager." At the time Ken was working on Titan Missile bases for PDM. "I spent two months at Des Moines home office then moved to Pittsburgh and spent six months on coordinating the construction work plan and fabrication details." He noted that the first four sections were built in the Pittsburgh shop. After that all the remaining sections were fabricated in Warren, Pennsylvania. Ken moved to St. Louis in 1962, arriving at the construction site in December and began overseeing the Arch construction.

Ken described the method of surveying used to assure correct positions of each section of the Arch. A theodolite scope was used and Ken learned how to use this instrument in WWII doing surveys in Europe. Ken noted that with the theodolite turned to “four positions” that their team was able to accurately place each section within 3/16 inch and convinced our (PDM) engineers and then NPS that they “had a pretty good handle on it.” A local engineering firm by the name of Richardson and Garden was hired by PDM to do the survey control work.

Ken described one of the surveyors. “One of those young men had blue eyes from Penn State, he was here during the 40th anniversary in 2005 and a girl came up to me and asked about old blue eyes and “why he is not here today? I want to meet him,” and I told her he’s not so young any more, he’s damn near as old as I am. All of them surveyed in the middle of the night. In the winter both legs had the same temperature. At quitting time Eldon Arteaga came down, he had a jeep and we had races in the mud or snow around the Arch until it was late enough to survey. Sometimes it was too windy to get a good line.”

Ken also noted that he helped PDM with developing the construction sequencing and project work plan. PDM had an engineering design group internally that were involved in developing structures for nuclear power plants, buildings and bridges around the country. Cables and guy wires were shown in the original plans for stabilizing the legs during construction. The initial approach was to use two 600 foot derricks to build the Arch. He noted that “guy lines would have had to been run out into the river to support the derricks but he COE objected.” The concept of using creeper cranes evolved.

The use of a strut at the 300 foot level to stabilize the two legs of the Arch was a PDM idea but it was originally conceived as a hinged strut. This was taken out which gave the engineers an opportunity to “weigh” the closure—very similar to closures in bridge structures. Hydraulic jacking while placing the strut helped the engineers to determine the closure pressure at the top of the Arch when the last section was placed. This was calculated and agreed to and was ultimately within 5 percent of estimates.

Ken described section erection procedures and how position was checked prior to welding. He remembered that at the north leg at section 46 or 47 that 3/8 inch had to be added to match up to south leg elevations. He mentioned again that the surveyors could obtain tremendous accuracy with their procedures. Typically at the beginning of the day an Arch section was lifted into place, fitted, shimmed and aligned. Then during the evening when temperatures were lower and wind had subsided surveyors would take measurements and compare the section position to engineering calculations. The surveyors would come onto the site at the end of the day to get the daily coordination then conduct the survey at around 3:00-4:00 AM. Joints and section positions would then be adjusted and final grinding and welding would take place the next day. All corners were 100 percent X-rayed while all stainless steel and mild steel welds were spot X-rayed for quality. All stainless steel field welds on the exterior were not ground as this was an architectural decision and helped establish the pattern on the skin of the Arch that was desired by Saarinen. Ken brought a brochure entitled “Layout and Erection Control of the St. Louis Arch” given at an ASME conference.

Ken mentioned that his position with PDM was “Project Manager” and his main duty was to coordinate erection procedures on site with the general contractor’s superintendent and was in charge of field work quality procedures. He said that it was considered to bring sections of the Arch down the river on a barge but winter work would have been difficult if not impossible so delivery by train was chosen.

Bob Moore asked, “What can you tell us about the concrete for the footings, and the installation of the 1-1/4 inch post-tensioning bars that would be used to stabilize the Arch as it was constructed?” Ken knew

much about the Arch design but not about the foundations as PDM was not involved in the foundation placement.

Ken went on to describe how the Arch sections were shop fabricated, shipped, assembled on site, and erected in place. The lowest first four sections were all entirely shop assembled as one large triangular section then shipped to the site and erected into place. After these four sections all remaining section from this level up were partially assembled in the shop and welded together in their final configuration on-site. The Arch sections from the first four sections up to approximately the 300 foot level were made in the shop as three panels—one for each side of the triangular section. These three pieces were shipped to the site and assembled on the ground where the corners were welded joining all sides into one triangular section. Pick points were welded at the inside intrados corners for creeper crane lift cables. These cables could be adjusted in length as each piece is different and fine adjustments could be made to assist in fitting the section into place. This section was then lifted into place and weld to the section below. All sections above the 300 foot level were made in the shop as three L-shaped pieces. These pieces were made so the field welds would occur on the sides of the panels not in the corners. Again these segments were assembled on site, welded and hoisted into place, fitted and welded to the sections below.

Steve Kelley and Ken Kolkmeier discussed the steel WF stiffeners used at the interior of Arch sections. The steel beams were used to brace sections until they were permanently set. Ken noted that the steel was left inside for the most part but was originally intended to be removed and salvaged. He also noted that this was an issue that was addressed during bidding. As erection progressed it became more difficult and costly to remove and was consequently left in place especially at the top of the Arch. It also was useful for workers to use as a platform from which to work. Steve commented that during our recent observations the HSR team had observed steel left in place and notched/torched in many locations to accommodate the tram system and stair components.

Ken reviewed the question: “What can you tell us about the welding process, and the inspections of the welds?” He stated that all welds were done to ASME specs and were X-rayed. Most stainless steel welds were done in the shop except as joining sections in the field. All exterior welds were argon gas/CO₂ shielded. All interior welds were hand welded while exterior had machine/jig. Ken said that the all the welders were very skilled and never complained.

The next question was reviewed by Ken: “What can you tell us about the creeper derricks and their operation?” Ken noted that the cranes and tracks were designed by Richard Gardens and fabricated by Pittsburgh-Des Moines. The derricks weighed 90 to 100 tons depending upon the “trash” you had on the platform. Ken had to stay on crews to keep clear and clean. They were load tested after installation. The creepers were color coded—any green-painted elements were stationary, and red-painted components moved. As the derricks jumped to a new section the back legs were adjusted so that platform stayed level.

A question was asked of Ken if he remembered any problems with placing sections when winds were high. Ken mentioned that if winds were above 25 miles per hour that no Arch sections were hoisted or moved. One morning tornado winds were forecasted. An Arch section being lifted to the 400 foot level was lowered back to the ground. The storm came through St Louis and broke some elevator cables. The crew had to stay up on the Arch leg and ride out the storm. Ken mentioned that the legs moved quite a bit until the strut was installed.

Ken was asked about job safety and precautions at the site. Ken noted that all the scaffolding all equipment was designed specifically for this job. All was custom and had to change as the crews worked up the legs of the Arch. It took about 2-1/2 months of planning and design for the scaffolding systems. Ken mentioned that they had only three lost time accidents during the entire period of construction—two were minor; the severe one was an eye injury from a grinder accident. He mentioned that PDM would fine workers for not wearing safety glasses and shoes. PDM made sure that safety equipment was always available for workers. The insurance companies predicted that 13 people would die on the job. PDM had a good safety history. Ken mentioned that “no other construction project over 300 man hours has had such a low accident rate.” This was due to “good supervisors and people who cared—all the supers on the arch were PDM employed for years some for 35 years. We were fortunate and worked hard at it.”

Steve Kelley asked about construction problems on the South Leg, and work on the North Leg came to a halt in 1963; the problem was in the post-tensioning bars. What do you remember about this? Ken discussed south leg construction problems with post-tensioning at splices and sleeves. Occasionally McDonald had some concrete get into a few sleeve and the steel rods became frozen and would not tension when pulled. The decision would be left to the engineer whether to grout the rod into the sleeve or cut out the bad area, install new sleeve and patch back. Sometimes holes had to be cut through the inner carbon steel walls to loosen up concrete in the sleeve. The concrete was patched and a patch placed over the carbon steel. Ken could not remember the sections where these patches occurred. Of course X-raying of these sections was difficult if not impossible because concrete interfered with film plates. Ken mentioned that many of the welders were from Titan II missile sites in Arkansas.

Ken described the typical construction sequence for erecting a section of the Arch. A section would be lifted and positioned into place/braced and tack-welded. At night after temperatures cooled and stabilized the survey crew would cross-check location with engineering calculations. The next day any final adjustments would be made on fit and the mild steel sections would be welded into place. These would then be X-rayed and then McDonald would install the PT rods. All the final exterior welds would be completed which would take approximately 2 to 2-1/2 days. Final X-rays would be taken and any repairs made. Concrete would be placed and allowed to cure for 24 hours or as soon as 4000 psi is attained the post-tensioning would take place. The leg section would again be surveyed to make certain that the concrete pour/tensioning did not affect the position or move the structure. Ken noted that Ted Rennison and Bob Moore (of the National Park Service Construction Division) were present at all pours and said that McDonald did a “wonderful job” on all the pours. They were very careful not to beat up the Arch sections with the concrete bucket, etc.

Ken was asked about the contributions of Dr. Hannskarl Bandel from Severud’s office. He described Bandel as a “tremendous” engineer who realized that what you design may not always be what you get in the field. He worked hard with the construction team to solve issues on the site for the best of the project.

The issues/complaints about wrinkles in the stainless steel plates were discussed. Ken noted that section 46 on the north leg buckled a bit as they were working it into place. PDM ended up taking it down and reworking it. He also mentioned that “anyone who expected that it (the 1/4 inch stainless steel panels) would have no wrinkles did not have a very good instruction in strength of material because when you weld on the back side of 1/4 inch material you get distortion from the welding.” Ken noted that Dr. Bandel helped with the solution for section 46 and that holes were cut in the top struts and the section was filled with concrete. The south section 46 was also filled with concrete so there was symmetry.

The closure of the Arch was discussed with Ken. He mentioned at the time the Mayor wanted a bright sunny day for the closure ceremony and wanted to bring in school kids for a picnic. PDM said no due to safety concerns. "Absolutely we were going to close that arch the minute we could." Ken said that the legs would deflect up to 30 inches in the hot October sun and it was estimated that the jacks to open the Arch for the final section would required 625 kips of force [1 kip is a kilo-pound, i.e., 1000 pounds]. PDM got about 624 kips when they "kicked it open" or very close to that Ken said.

McDonald Construction Company filed for bankruptcy around the time the Arch was finished. Ken provided a bit background on this issue. He mentioned that they were involved in a very large public sewer project that did not work out and that caused the filing for bankruptcy and that it was not the cost overruns on the Arch project.

A firm named Fairchild did wind tunnel testing on a model of the Arch. Ken mentioned that they had built a 6.3-foot high plastic model of the Arch and had it on site to check clearance for the derrick and other dimensions.

At the time of closing Ken said that the centerlines of the north and south legs were within 3/8 inch of alignment. During the heat of the day they would deform up to as much as 30 inches. The Arch was designed to sway 18 inches in a 150 mph wind. Ken said that at 80 mph they would get 3 to 4 feet sway in the legs before closure, and this is why PDM wanted to close the Arch as soon as they could.

Steve Kelley asked Ken about the issues surrounding the interior micro-climates of the Arch legs. Ken confirmed that absolutely there were times when fog would appear inside the legs and there would be episodes of condensation. Steve showed Ken photographs of recent inspections where there was corrosion of the heads of threaded rods connecting the stainless steel outer stainless steel plates and the inner mild steel plates. Ken agreed that any temperature observed on the outer skin could be conducted through to the inside via these rods and other plates. Ken described the methods of how the inner and outer plates are joined. Above the concrete fill the stiffeners are stitch welded; within the sections filled with concrete the stiffeners are held by studs welded to the stainless steel out layer. Ken noted that the original plans had continuous stiffeners but his was impossible to do because each section of the Arch changed as it progressed upwards-so how could they be continuous? This was changed in the addendums and eliminated to the method described. These details were reviewed on a set of original contract documents which Dan Worth had on the conference table. Ken also discussed the value adjustments made after the bidding process which included reducing the stainless steel plate thickness from 7/16 inch to 1/4 inch and the mild steel plate from 1/2 inch to 3/8 inch. He said this made the engineers "a bit nervous."

Steve Kelley noted that in a recent study that WJE and BVH conducted for NPS that several areas of staining were mapped and analyzed. Ken said that some stains may be a result of equipment abrasion against the stainless steel or residue from construction and the clean-up process. Ken and Steve had a lengthy discussion on how to access the Arch skin at these areas via crane or rappelling. Ken noted that the contractor used the derrick platforms to clean the stainless steel surface as they dismantled the system working down from the top of the Arch. Ken showed a photograph of this process. Ken allowed JNEM to scan these photos for their archive and records.

Ken described the crew which did the majority of the stainless steel welding. From 300 feet and up the men/crew did not change. It was a 40 hour a week job—"a good job"—and the men had complete confidence in each other. This crew moved from one leg to the other to perform the stainless steel welding of the sections.

In further discussing the revisions to the steel skin thickness and the resulting halos on the plug welds, Ken said that he was not too involved in these decisions as they were made in the shop and not on site. The topic of whether there might be some water “trapped” between the inner and outer skins due to condensation or leaks at the upper elevator ventilation grills, Ken noted that each panel above the concrete had continuous steel stiffener plates at the top and bottom of each section that would not allow water to migrate between panels. He also mentioned that the elevator vents were installed after he left the job site on December 31, 1965. The vents are welded all around. KJK initials were welded inside one of the vents before he left. Ken went on to say that he felt that the inside of the Arch always felt damp or moist. As work progressed beyond the 300 foot level, Ken installed doors at the base of the arch to seal off the bottom part. The Arch legs acted as huge chimneys and when it was windy, blasts of air would rise up the legs. The doors helped to prevent this and helped with crews working inside the legs.

Ken mentioned that soon after the lower sections of the Arch had been set and concrete pours were made that the level of the pour was changed from the original specification. The top of the pour was lowered approximately 12 inches below the top of the section instead of level with the top of the Arch section to allow workers flexibility in working with the post-tensioning rods and to give flexibility in setting the next Arch section.

Southwest Ornamental Iron fabricated and installed the interior stairs and tram. PDM allowed them to use the derrick to hoist and drop beams/sections into the Arch legs whenever they were not hoisting Arch sections. Ken noted that the stairs fit very well and this was an indication that the Arch was being constructed correctly.

Bob Moore noted that during the Fortieth Anniversary in 2005 one of the local news people asked if the Arch was rusting. Ken replied that “stainless steel does not rust. If it was scratched with construction equipment, if the harness had rubbed against there and later cleaned it up and over 40 years there might be some residue. I can tell you right now that the parent material is not rusting, that is my feeling. If I tell you how to get up there, then you’d have to put me on your payroll! There are cranes that do that, but it can take two months to assemble, but not with the Museum under there. You better have that on some pretty firm ground (for the cranes). There’s a guy in Chicago, son of an ex-Nooter [Corporation] president that sells or rents equipment that jacks you way up, and I can give you that name if you want to talk, but it’s a scary option, 400 feet in the air out over the measurement. I don’t know whether it will work or not.”

Ken described the different stainless steel welds and the possible impact upon the accumulation of dust and streaking. He diagrammed the ideal weld was one that had a back up bar behind the stainless steel. After the section had been tack welded in to final position a track was attached to the outside and the joint was ground. The first weld pass would fuse/join the stainless materials and back up bar together and would be shallow or below the exterior surface plane. The next pass/final pass would complete the weld with a nicely concave weld. The welds were not ground but left intact. If the joint was widened due to elevation adjustments, the weld could become wider taking up to two welds for pass. This would result in a final weld that had a double concave shape. Ken noted that “this little bit of weld will catch dust” and when wetted could cause the discoloration or streaking. He also said that the rippling may be a function of the different rate of expansion of stainless steel and the mild steel stiffeners. Stainless steel expands at a 50 percent greater rate than mild steel and if both sides are tied together by the stiffeners and have different temperature gradients then you “would expect ripples at different moments.”

Steve Kelley asked “in 10 or 15, maybe 20 years if the NPS needs to clean the Arch, what’s your idea?” Ken responded that “the Arch, Eero Saarinen said, would be maintenance-free. He noted that because of the shape and geometry that harnessing around the Arch would be necessary to pull someone back into the surface to get up close to work. “But that’s a nervous thing to do.” He also said that “I know have a solution for this area. I’ve looked at this area around the strut attachment and that is where I think people have said it’s rusting, discoloring, at the spot of the strut.” Ken went on to describe how the strut was connected to the Arch section 22 and 23 during construction. He diagramed a detail showing how a steel connection plate was welded to the inner steel plate and extended through the stainless steel skin and had a plate for bolting and connecting the strut structure. When the strut was removed the steel connector was torched off just below the stainless steel skin and a piece of stainless steel patched was welded into the hole and ground smooth.

Ken shared a copy of the July 4, 1965 insert section of the *St Louis Post-Dispatch* that had photographs of the project. Ken also shared several black and white photographs of fabrication of Arch sections including assembly of sections (one section above the 300 foot level and one below), a photo showing the alterations to Segment 45 to allow grouting, and a photo of a section below the 300 foot level being grouted. Scans were made of these by Jennifer Clark and digital copies were given to Dan Worth and Steve Kelley.

Interview was ended at 12:18 p.m. for lunch.

Interview was reconvened at the Gateway Arch at 2:00 p.m.

Ken Kolkmeier was taken up in the freight elevator to Arch section 38. Chuck Kalert noted that almost eight years ago the heating coils failed in both legs of the Arch and steam was released into the legs. There was approximately 8 inches of water condensate in the tram pits as a result. The coils were replaced but are starting to fail again.

Ken walked up to Arch section 35. He commented that he had not been inside the Arch legs since 1965 and that the condition to him looked “real good”. At this section Ken pointed out the hoist plated and how they had been removed from this section. He also pointed out the plug welds and the good alignment of the panel sections.

Ken returned to the ground level via elevator. Steve, Al, Dan, and Bob walked down the stairs. Steve Kelley noted an unusual bolt pattern at section 64. Ken K commented when all convened at the bottom that this could have been where heavy beams were inserted to handle the lower end of the creeper cranes structure.

Meeting adjourned at approximately 3:15 p.m.

Interview with Bruce Detmers – Transcript with Additional Notes

The following oral history interview was conducted with Bruce Detmers, architect with Eero Saarinen and Associates, by Deborah Slaton and Michael Ford of Wiss, Janney, Elstner Associates, Inc., on April 1, 2009. The interview was conducted at the Yale University Saarinen Archives in New Haven, Connecticut. The questions were provided to Mr. Detmers prior to the interview, and at the interview he provided prepared notes in response to some of the questions. The interview questions and responses follow, with supplementary notes provided by Mr. Detmers indicated in italics following the questions below.

[Direct transcript of interview and copy of supplementary notes provided by Mr. Detmers will be submitted with final report, under separate cover.]

Deborah Slaton (DS): This interview is with Mr. Bruce Detmers, architect with Eero Saarinen and Associates, and is being conducted by Deborah Slaton and Michael Ford of Wiss, Janney, Elstner Associates, Inc., on April 1, 2009, at the Yale University Saarinen Archives in New Haven, Connecticut.

Background and Arch Construction

1. First of all, Mr. Detmers, could you tell us how your interest in architecture began? What aspects of architecture intrigued you (structural engineering, design, construction)?

My father was involved in the construction industry. I worked while a student in high school remodeling houses. I also worked for a contractor while in high school building houses. I worked mostly as a carpenter. My initial interests were in construction but changed to design while in college. While in college, I worked as a designer for a small office, Errol Clark. I worked for Smith Smith, Hinchman & Grylls, and Earl Meyer and Associates, prior to joining Eero Saarinen and Associates.

Bruce Detmers (BD): I went to school at Lawrence Institute of Technology in Detroit. Upon graduation, I was inducted into the Army and I was classified as a civil engineer and I worked at Fort Belvoir, Virginia for the Corps of Engineers. It was the Special Projects Division for the army. We studied the effects of atomic weapons on mines and foxholes. After leaving there they wanted me to stay on in the research and development laboratory but I was not interested in that kind of thing. After graduation, I worked for Errol Clark, in fact, I worked for Errol Clark as a designer while I was in college, doing conceptual designs for buildings, primarily churches, which he did, and some doctor's offices. I worked at Smith, Hinchman & Grylls and Lane, Davenport, and Meyer. That occurred between 1950 . . . I graduated in 1952, got out of the Army in 1954 and I started working at Eero's office in 1956. A good friend of mine, Leon Jankowski, was working at the office and I worked basically in the working drawing section of the office. There was a design section and a separate working drawing section, in which we were responsible for developing contract documents and specifications for the buildings. I worked on a women's dormitory at the University of Chicago; I worked a little bit on the law school at the University of Chicago; and . . .

DS: I think you said you worked on Vassar College.

BD: Vassar College dormitories.

2. What was your background experience prior to working with Eero Saarinen and Associates?

After college, I was drafted into the U.S. Army in August 1952, two months after graduation. I was classified as a Civil Engineer. I served two years active duty, 1952 to 1954. I was assigned to the Engineer Research Development Laboratory at Fort Belvoir, Virginia. I was involved in the Army's study of the effects of atomic blast on land mines and fox holes. I was present during several atomic tests at Frenchman's Flats in Nevada. I collected data during the tests and returned to Fort Belvoir preparing reports of the test data until being discharged from the service.

3. At about what stage was the Arch project when you first started working on it?

Eero Saarinen won the competition in 1948. I joined the Saarinen office in 1956. Congress at that time had not appropriated funds for the project until later in that decade. Some models of the Jefferson Memorial were in the office but there was little activity on the project at that time. I worked on two of Eero's projects, a dormitory for Vassar College and a University of Chicago Residence Hall. There were two departments in Eero's office, the Designers and the Working Drawings department. The Jefferson National Expansion Memorial was close to being completely designed at the time I was involved. I was involved in the final design development phase of the project, including the site plan, changes to the levee, the relocation of the train tracks through the site, and the Grand Center steps. These were the initial projects contracted in the development of the Memorial. I eventually became project manager, involved in the preparing contract documents for these initial projects.

DS: And from your office you could look out the window and could see the model for the stairs at the Arch?

BD: Yes, I could, when I started working at the office I was put in the new section of the expansion of the office on Long Lake Road and it was a good spot to look out and a wooded area behind the office and there was a major meeting with the . . .

DS: With the Park Service?

BD: National Park Service and I think the City of St. Louis came out and so on. It was always a major time and it was at a time when they were talking about site development, also the museum and Eero had hired Jo Mielziner, as I remember, to do the interior work on the museum.¹ He was going to use a sort of subliminal kind of presentation rather than the display cases you normally see in museums so it was a big departure from what museums were . . . from what NPS was doing for museums at the time.

4. What was your job title on the Arch project?

Project Manager, Job captain, I represented the office in dealing with the National Park Service during the construction period. I was involved in the answering contractor's questions during the

¹ Jo Mielziner (1901–1976) was a famous Broadway set designer known for his original designs for the productions of *The Glass Menagerie* (1945), *A Streetcar Named Desire* (1947), *Death of a Salesman* (1949), and *After the Fall* (1954).

bidding process and represented the office during construction phase. I made frequent trips to St. Louis from our office in Connecticut.

DS: So, when you first started working on the Arch, what were your initial responsibilities? Were you involved with the site planning or the structure?

BD: First thing we started to do was work on the levee and the relocation of the railroad, two railroads. One right on the levee and the other one was an elevated railroad which was extended into the park and we had to build tunnels and so on for that. We hired special engineers to help us with the design of the railroad track and so on.

DS: I think that you mentioned that the first time you went to the site it was just a wasteland of . . . the buildings were gone?

BD: The buildings had been torn down. I think that the NPS had selected the site and the buildings had been torn down and the site had been sitting for years as a big vacant . . .

Mike Ford (MF): I have photographs here. Is this something similar to what you were seeing or where would you say this photograph is reflected in that process?

BD: No, this was long after the site development. Drawings had been done, you can see the elevated track in this drawing. I don't know if you can see, this drawing isn't clear enough, but there is a railroad track that runs along the levee. What we did was make some small retaining walls toward the river side of the levee and then, you can see on this drawing where the new railroad had been put through and see the tunnel at the center of the site and you can see that construction started at either end of the site. One of the things that we had to do was to build a flood wall or a way of attaching the flood wall, I should say, to the overlook buildings to prevent water from getting into the site and surrounding areas.

MF: This photo was probably right after Phase I of the levee building?

BD: Yes, I would say this was after Phase I and must have been, I would say, probably about 1959, maybe . . .

MF: And when you arrived, there was a lot more debris, you were saying . . .

BD: Oh yeah, there was a lot more debris . . . You can see what it looked like . . . they had to put some parking at the east end of the site . . . what I remember about this site too is that they made licorice in an area toward the Eads Bridge side and every time I came to St. Louis I could smell the licorice and also the Monsanto . . . there were a lot of chemical smells from Monsanto. That might have been from East St. Louis.

DS: Do you like licorice?

BD: Yes, I love licorice. Every time I eat it I think of St. Louis.

MF: I know one thing you had mentioned is the tall buildings in the area, that there was a correspondence about building height in the surrounding area, as far as exceeding the height of the

Old Courthouse. Were any of these buildings causing a concern with Saarinen as far comparison with the Old Courthouse?

BD: Not these buildings. The old buildings were not a great deal higher than the top of the courthouse. There were a few buildings that were built that were quite a bit higher.

DS: Before the Arch was built or before it was completed, Saarinen and William Wurster provided a report to the City to consult on zoning and I think that was the report where they made a strong recommendation to limit the height of all buildings along the line with the courthouse. So I think up until that point things were pretty much at that height or below but since then, there have been some taller structures.

DS: We wanted to ask your job title on the Arch project.

BD: You know I think, at the beginning it was probably what you would call a job captain, basically responsible for developing the contract drawings. After the Arch got under way, I think during that period Joe Jensen was the project manager or John Dinkeloo. John Dinkeloo was the partner in charge; Joe Jensen came on site. Joe did a lot of estimating and helped Kevin [Roche] come up with the design, helped Kevin by making estimates of some of the creative ideas he had, and John Dinkeloo was partner in charge. He was very clever at construction concepts and the interesting thing, as I recall, is that Eero had died in 1961 and our office was kind of planning to move to Connecticut and the Arch drawings themselves were pretty well complete but the contract for going out and getting bids for work had not been completed and so in the middle of . . . some of us had already moved out to Connecticut and some were still in Detroit, so all of the concerns we had in bidding the job and moving and so on. I guess, by the time we moved to Connecticut, we had found the contractors; we had established the low bidder and so on. The bidding process was kind of a marvelous time and I remember the contractors coming in and the way the government assigns contracts is that they come in and ask questions about the contract documents and a lot of those questions wound up on my desk and I would give our interpretation or tell them what the intent of the contract documents were. And it was in the Old Courthouse kind of setting; George Hartzog was a very forceful kind of a person so he definitely led the group. When it actually came down to answering the questions in writing, I worked with George quite carefully to answer the questions that would be in accordance with the intent of the contract documents. So it was an interesting time bidding it.

DS: Did you feel that the contractors thought this was a very unusual or challenging structure?

BD: Absolutely, it really came right down to it that whoever had the best way or best concept for erecting the Arch, they would be the successful bidder. Because there would be so many ways of trying to support a structure like that going up. How do you do it? 630 foot high Arch. It was a major task and there were questions about how much load the Arch could hold while being erected. Therefore, our engineers got involved in analyzing the winning design or concept for erecting the Arch.

DS: Did you feel or did others in the office feel that this was a particularly challenging project from your perspective as well?

BD: I think it was. I think definitely it was. One thing you might want to think about a little bit is that Eero had died and he was the principal designer in the office. Joe Lacey was really the only other registered architect with Eero. John Dinkeloo and Kevin Roche were not. I think John was a registered Civil Engineer and Kevin, from Ireland, was not registered at the time. That made another concern for the office. They too were interested in their own projects. They were thinking ahead about where the office was going. The Arch was important, extremely important, to Kevin and John and Joe, but they were also thinking ahead to getting new work so that the office could continue. So, it was a distracting and interesting time. I was about twenty-eight or thirty years old at the time. It certainly was an honor. I remember one thing that a professor at school told me. He said, "Always work for the best firms that you could. Don't work for companies that are going to give you bad habits." So working for Eero, I was going to develop good habits. I wanted to go to Cranbrook Academy of Art at the time. I had two children, but I still wanted to go to Cranbrook to further my education. I remember coming into the office and John [Dinkeloo] told me; "Look, Bruce, I can teach much more than you are ever going to learn at Cranbrook Academy of Art." He persuaded me to come to work [in Saarinen's office].

5. Could you give us a brief overview of the characteristics of the Arch, and how its shape and form evolved? How did structural stability affect the design process?

The Arch is a unique structure and form which is immediately associated with St. Louis. The form of the Arch distinguishes it from any other structures in the world. The initial shape of the Arch, I have read, was determined by Eero bending a pipe cleaner while having a meal. The development or refinement of the Arch was by hanging a chain. This created a shape that was too flat at the top. There were attempts made to weight the chain to alter the shape but that was not successful. The final shape was determined but a mathematical formula which Dr. Hannskarl Bandel, structural engineer with Severud Elstad Krueger, suggested and we used to define the final shape of the Arch. The form of the Arch was the determined by Eero Saarinen as a sculpture. The structural engineering accommodated the Arch design.

6. Could you link the design of the Arch with Eero Saarinen's interest in and study of sculpture?

After several attempts to define the Arch from as previously explained, we used the mathematical formula to describe and dimension the Arch. Eero reviewed the large drawings that were plotted and approved the shape. Eero made several models of the Arch from very small ones to the final model which was about 8 feet in height. Eero was a sculptor and was constantly involved with the final form of the Arch.

7. What were your duties and responsibilities on the Arch project before construction began?

See answers to question 3 and 4 above.

8. What were your duties and responsibilities on the Arch project during the construction process?

See answers to questions 3 and 4 above.

Design of the Arch

9. Did you ever hear discussions about the 1947–1948 competition for the design of the Arch, or how Eero Saarinen came up with the idea?

Yes, see answer to question above.

10. Did Saarinen ever talk about the other competition entries?

Not to my knowledge. Bill Eng joined Eero's office. Bill placed high in the competition for the design of the Jefferson Memorial.²

11. It seems that Eero Saarinen tried new and daring concepts in the design of each of his buildings, which posed challenges to engineers and people from other discipline in actually getting the structures built. In the case of the Arch, experts such as Hannskarl Bandel stepped in to make the structure feasible. What can you tell us about this process, first, in the way that it generally worked on several Saarinen buildings, and second, how it worked on the Arch?

I recall that Severud Elstad and Kruger initially came up with a structural frame concept for the Arch, which involved placing the skin of the Arch on a steel frame. This concept Eero rejected. Eero wanted the skin of the Arch to contribute to the Arch's structure. The other structural criteria were that the Arch should last one thousand years. The engineers revised the concept of the structure; the outer steel skin of the Arch was stiffened on the thinner side to make it a supporting member. The final structural design can be described as inner and outer steel plates structurally tied together. The space between the inner and outer plates is filled with post-tensioned concrete which is connected structurally to the massive foundation. The upper inner and outer plates of the Arch are fastened together with steel members without the post tensioned concrete.

DS: We saw upstairs in the Archives some of the pictures and models of the different configurations that the Arch took before it was refined into the final shape. Can you talk a little bit about that process? Was that still going on when you joined the office?

BD: Yes, when I started working on the Arch, the shape was not defined dimensionally. How to dimension this thing? As I recall, we hung a chain—you probably heard this story many times—the way I looked at it, when we hung the chains, the top of the Arch came pretty flat and sort of rounded almost. Then what we did is we took some rubber and we cut it like this so that it would be heavier at the bottom when we hang this thing. Because of the weight, we thought that this would pull in more and give us more of the shape that we were looking for. And it did to some extent. And then we were trying to weight the chain with various weights to try to come up with the shape we wanted. We came up with a formula which is on the drawings here. There is, AR1 [referring to drawing], which was the basis of the calculations. Actually Hannskarl found that formula and I don't know where it came from. I think, it must have come out of some German book because I have never seen it anywhere.

DS: The formula for the shape of the curve. Interesting.

² Bill Eng's entry for the Jefferson Nation Expansion Memorial design competition placed second, just behind Eero Saarinen's.

BD: I have never seen it. And there have been some mathematicians I talked to who actually are involved in this kind of research and they have never found it either. So, I don't know where that formula came from, but it worked for us. What we did, we made a drawing and had it blown up so that it was, I would say, it must have been 10 or 12 feet high. So, you could really see the shape. After we had drawn it—and it was a lot of work to draw this, as you can see by the numbers—and have Eero come in and look at it. I don't think he looked at it for more than five minutes and he said, "It's fine."

DS: Really, after all that!

BD: I was so relieved!

DS: So the models and working with rubber and so forth to get the concept of the shape, that was all going on. Then this formula was brought in by Mr. Bandel and the formula actually met the exact need of the curve? That is amazing.

BD: I think Eero made a model of this after to check it but he told us to go ahead with it. So, you don't know how relieved I was at the time. You know you have to go into all this work and try to figure out how to . . . , looking at a blank piece of paper and say okay, how are you going to define this? To me, it seemed like a big deal. It was a big deal.

MF: You mentioned in the Archive, the grand staircases and retaining walls followed this weighted catenary shape. Was it the same formula or was it more of a typical catenary.

BD: No, it was more . . .

DS: You mean that exterior stair and site walls, they were all parabolic. That is very interesting.

12. Could you describe Hannskarl Bandel for us?

Hannskarl Bandel physically had dark brown curly hair, solidly built, friendly personality, but tough negotiator with contractors and other engineers. He was unmarried, lived with his girlfriend Irmtraut Sitter. They lived in New York City and had a house in Pennsylvania. Their dog's name was Pox. They were both Germans; we became good friends as we traveled to St. Louis and other places together over the years. Hannskarl served in the German U-Boat Service during the Second World War.³ He once said that Germany should have never given up to the Allies. One funny story he told me, I recall, was that the bank asked him if he served in the military during the war, and suggested that he may be entitled to a G.I. bill mortgage.

DS: You were telling us a little bit about Hannskarl Bandel and his response to the misalignment of some of the rods.

BD: Yes, you can see in this next photograph [*indicates photocopy provided by WJE for interview*], you can see the amount of steel that he made them put in for that. One thing that I was very disturbed

³ Although reference is made to Hannskarl Bandel as serving in the German U-boat or submarine forces, it is known that Bandel actually served as a junior officer aboard the motor torpedo boats (also referred to as Schnellboots or S-Boats) stationed in the North Sea.

about is that Hannskarl wanted this thing done, as far as the engineering is concerned, the way he wanted it done. He told me at a time when we were not having a meeting or anything, just having a conversation, he said he didn't really need all this steel, he made them put it in because he wanted to make sure that they understood that he wanted it.

DS: Oh, that he was in charge. Could you describe him a little bit? He sounds very forceful.

BD: Well, Hannskarl, I would say was a little older than I am, because I was not in the Second World War and he was. He was in the submarine forces for the Germans and patrolled the waters off of England and Hannskarl, at one time, told me "We should never have given up!" But we became good friends because we would travel to Pittsburgh and out to the Arch. He was from New York. He came to my house a few times. He had a dog and his name was Pox, which he said meant peace. He had a girlfriend, her name was Irmtraut Sitter.⁴ She was from Germany and they lived together quite close to the office at 415 Lexington Avenue. I think that building is gone now. Hannskarl spoke with a decided German accent and his partners in the office were Fred Severud, who was really the partner in charge as far as the engineering was concerned. Werner Gottschalk was another good friend and was responsible basically for the specifications as far as the structural engineering was concerned. One of the things we got into a problem with the Arch was the radiography and the testing of the welds. The boiler code which he cited in the specifications required that the contractor pay for the laboratory testing of the welds. The boiler code required the testing of the welds. And normally, the owners hire a testing laboratory to test the welds. So there was a conflict in this respect.

DS: You mean, normally the owner would pay but he specified that the contractor would pay? The boiler code required the testing?

BD: The boiler code required the testing.

DS: But the code didn't say who had to pay for it, correct?

BD: The interpretation that Werner Gottschalk put on it was that the contractor would pay.

DS: But that wasn't the typical interpretation?

BD: That was the dispute that we had. But the other thing I remember, getting back to Hannskarl, was that he bought a house in Pennsylvania that he would use on the weekends. Betty, my wife, and I went out there a few times. I remember Hannskarl telling me that when he was looking for a mortgage, they asked him if he had been in the service and he told them he had. They said you might apply for the GI Bill.

DS: Did he apply for it?

BD: No.

DS: That would have been interesting.

⁴ Hanskarl Bandel later married Irmtraut Sitter and they resided in Colorado.

BD: So that's a little bit about Hannskarl. I would say he was a brilliant engineer and if there were ever any questions, he wouldn't equivocate or step aside; he could definitely answer these questions in a very specific way.

13. Major changes to the original plan came as production geared up in 1959; what can you tell us about this? [The museum would be underground, beneath the Arch; access would be by ramps; grand staircase and the railroad tunnel; Old Rock House eliminated from the plan; Saarinen refining the shape of the Arch itself]. In the final phases of design, what changes and alterations were being made?

Eero Saarinen during the initial design stage of the Arch was to be 590 feet in height. The final design stage the National Park Service accepted Eero's recommended that the Arch be 630 feet in height. I believe that the reason had to do with the height of the buildings proposed for the City of St. Louis.

The structure for the underground museum, theaters, corals [sp.?] for accessing the conveyance system tram, and with ramps was built in accordance with Eero's design. The museum finishes were done with another architect. Eero hired Jo Mielziner, New York stage designer, as consultant to do the museum interiors; however, the National Park Service selected another interior designer for the museum. Joe Mielziner's presentation did not go well with the Park Service.⁵

The Jefferson Memorial was built in stages as funds were made available by Congress. Stage one was the improvements to the levee. The second stage was the retaining walls, relocation of the elevated railroad tracks and overlook structures [instead]. The third stage was the Arch and visitor center structure. The grand center steps came later and were not built in accordance with Eero's design. The conveyance and elevator system in the Arch became a separate contract with the Metropolitan Transit Co. in St. Louis. This system was developed under design by Eero Saarinen and Associates, and Dick Bowser as conveyance consultant.

Eero's priority was that the Arch was first over the landscape and site work. The Old Rock House was important to the Park Service but funds were not adequate to rebuild it at the time the outlook structures were constructed. There was some discussion regarding the location of the Rock House.

Eero was always rethinking his designs, he was constantly studying and proposing changes as the work progressed. Nothing was sacred until it was built. Many changes were proposed and adopted during the course of the project.

The layout of the walkways, the pedestrian bridges over the highway connecting the site of the Arch, and the Old Court House, and on-site parking garage were developed during the final stages of the project.

⁵ Although reference is made to Jo Mielziner, it is known from oral histories conducted with George Hartzog, Bill Everhart, and Kevin Roche that Charles Eames was the contracted museum consultant. Eames' presentation in 1960 in the Saarinen office, which was attended by Hartzog, Everhart, and Roche, was seen by National Park Service representatives and rejected by Director Conrad Wirth.

14. Low funding meant that the project was pared down to the Arch, Visitor Center, and Museum. Later, funds only covered the Arch itself. What decisions did you have to make to alter the project to keep pace with evaporating funds?

Stone was omitted from the retaining walls. Interior of the museum was delayed. Landscaping was delayed.

15. What provisions were made for earthquakes?

The Arch foundations are deep within the rock on site, within the Arch structure is post tensioned providing stability to the Arch structure. Wind tunnel tests and other precautions were considered during the design phase.

16. Could you tell us about the genesis of the idea for an underground museum beneath Saarinen's Gateway Arch?

Eero's 1948 competition proposal was a below ground building for the museum.⁶

17. Could you tell us about the curved retaining walls and making those into hollow museum spaces?

The Park Service considered using the outlook buildings as additional museum spaces. Funding was unavailable to proceed with these buildings. The cross section of the retaining walls is similar to the Arch geometry as is the stairway leading to the outlook buildings. The cross section of the proposed grand center steps was to have the same configuration.

18. Could you tell us about Eero Saarinen's death, and how it affected construction of the Arch?

Eero had made the discussion to move east as his wife Aline Saarinen was the art critic for the New York Times. People in the firm were moving east from Michigan to Connecticut when Eero died. Eero's partners decided to continue to move as many projects were underway at the time. Many of our consultants were New York City based. The contract documents for the Arch were complete or near complete at the time of Eero's death. The documents were finished and sent out for bids after Eero's death. Eero's death did not impact the construction of the Arch as the design phase was complete at the time.

19. What else was designed for the Arch project and by whom after Eero Saarinen's death?

The museum interiors, dioramas, display cases, theaters. Aram Mardirosian, former associate of the Saarinen firm was the museum designer.

Basically the work of the Arch, levee improvements, elevated train relocation, and outlook buildings were done with the Saarinen office as architects. The work done after 1965 was accomplished by the Park Service staff and others.

⁶ Saarinen's 1948 competition design included an arcade with some exhibit spaces landscaped into the ground, which may have given the impression that certain features were to be built underground. However, the museum buildings were to be built above ground.

20. Could you tell us about the contributions of Joe Jensen?

Joe Jensen, engineer, estimator, and worked closely with Eero on many projects including the Jefferson Memorial. We became good friends; he moved with the office from Michigan to Connecticut. Joe Jensen was my mentor and worked to have me represent the office with the Park Service as the design progressed. I was involved in the development of the contract documents. Joe and John Dinkeloo represented the office during this stage. Joe moved from Eero's office to a position with the National Park Service when George Hartzog was director after Eero's death. The date of Joe Jensen's move to the Park Service is unknown to me now.

21. George Hartzog pushed for the construction of the underground museum, despite the wishes of the Eastern Office of Design and Construction, which decided not to include an excavation of the museum site in their plans. Do you know anything about this fight?

Not much, although, the scope of work for the Arch construction included the excavation and structure for the underground museum.

22. What can you tell us about dealing with the National Park Service as a client?

The Park Service was a good client; there were two Bobs—Bob Smith and another Bob—that were contacts in addition to a construction division representative on site during the bidding period early in the construction phase. There were some disputes; we generally worked together well. We were never, however, used as architects on future work for the Park Service.

23. Could you tell us about the interior finishes of the Arch museum and observation deck area? What was originally designed and what was eventually constructed? Knowing the amount of pedestrian traffic the observation deck and interior areas have seen, how do you feel the interior finishes should be treated?

We were not involved in the interior finishes of the museum (see answer to previous similar questions). The interior of the observation area at the top of the Arch was done in accordance with Eero's design as are the Corel [sp.?] staging areas for the conveyance system. I have not seen the staging area and observation deck in more than twenty years. One hopes that the building is well maintained.

Transportation System

24. What can you tell us about the "Sky Ride" designed for the Arch? Dick Bowser?

The elevator industry would not address the design of the conveyance system. Dick Bowser's father was the inventor of the pigeonhole system for parking on small lots in New York. Dick Bowser was contacted and requested to suggest a conveyance system to move people to the top of the Arch. He designed the tram and elevator system in the Arch using principal components used in the elevator industry. Dick was an inventor rather than a professional engineer. He developed a model of the conveyance system car and drawings used to bid the work. The contract drawings of the system were checked by a mechanical and structural engineer prior to the installation.

DS: The other innovation that I thought also was quite amazing was the transportation system with Dick Bowser. I understand from reading that he approached Eero Saarinen with this concept and then Mr. Saarinen asked him to submit a proposal. That was how they got together. Did you have an opportunity to meet him?

BD: To meet Dick Bowser? Oh yes, Dick Bowser and I worked together when You know, Dick's father was an inventor. As you said, he designed this pigeonhole system for parking cars in small lots in New York where you get a high density of cars. As far as I know, Dick did not go to college, he had no formal engineering, he was not registered as an engineer in any state. But, he had this marvelous ability to work on elevators. He worked in the elevator industry, I'm not quite sure what company he worked for, but he worked on normal elevators. I think, Dick has written his memories and he said that there was a meeting at the Park Service and the Park Service had to come up with a scheme or a consultant to design the conveyances. Dick came to the meeting and they sort of just popped this whole thing on him and he said he thought he could do it. He used the concepts of the elevator industry, he used their components, the drive on the top of the elevator. But, as far as the conveyances in these small little capsules that take you up to the top, keeping them leveled because the tracks are above you when you are at the bottom of the Arch and when you are at the top the tracks are under you so it goes through a complete cycle. Dick and I worked together because we had to make this thing shift in the Arch.

25. What were Saarinen's contributions to the capsules?

I do not recall.

26. During 1965, the Arch Transportation system lagged behind Arch construction and was \$318,000 in debt. Do you know anything about this?

No. I do know that bonds were sold to pay for the installation of the system and that the Saarinen office was offered to become part owners of the system in lieu of receiving a fee for the design of the system.

Construction

27. Who came up with the idea for Creeper Derricks?

The contractor, Pittsburgh-Des Moines Steel. Erection of the Arch was the contractor's responsibility.

28. How closely did your office work with MacDonald Construction once the building started?

Our relationship was typical; we were consultants to the owner. We had a representative on site during construction and processed shop drawings and meeting with the owner and contractor during the course of the construction. George Hartzog however maintained a very close relationship with Mr. MacDonald during the construction phase. Because much of the Arch work was subcontracted to Pittsburgh Steel we did have meetings with that company regarding technical details of the work.

29. How closely did your office work with PDM once the building started?

We had meetings with PDM on several occasions. We worked closely as the work progressed.

30. What do you recall as being the prominent issues during the construction of the Arch?

There was an issue regarding the contract with respect to inspection of the welding. This issue was resolved in accordance with the specifications cited in the contract documents.

31. What can you tell us about the concrete for the footings and the installation of the 1-1/4 inch post-tensioning bars that would be used to stabilize the Arch as it was constructed?

The post-tensioning bars were installed in a manner that additional reinforcement was required by Hannskarl Bandel, the structural engineer.

MF: This photo is the foundation and early stages. Is there anything that maybe strikes you or that you recall about this time?

BD: Well, as you can see by looking at this photograph. Basically it's the excavation for the museum. It's also from the end where you can see the starting of the excavation for footing for the arch. You can see that the top overlay is a kind of striated material loose but as you go down you see the striations are getting thicker. We went very, very deep on both ends of the formation for the Arch. Basically because of the way the conveying system worked, it actually had to come up through the foundation of the arch. So when you are in the corral area ready to get onto the conveying system to go up into the top of the arch, you are well within the footing of the arch. But, you can see that there is a lot of, I guess, limestone type rock here and the footings of the Arch are embedded deep into this material.

MF: In the next photograph, we have some of the post-stressed tensioning

BD: These are the rods coming out. There were some misalignments of these because they weren't exactly the slope that the engineer had anticipated. This photograph is of the exodus side of the Arch. These post tensioning rods you can see where they are capped off at the top and how extensive and how high they were.

32. Do you have any thought on pouring of the concrete between the outer and inner skins of the Arch legs? How did the actual concrete formula and pouring of the concrete differ from the plans and specifications? What was the reason for the change?

I do not believe that there was any issue regarding the placement of the concrete in the lower portion of the Arch. I do not believe that there was any change in the concrete mix that impacted the strength of the concrete.

33. What can you tell us about the welding process and the inspection of welds?

The contractor developed an automatic welding system which produced a uniform weld on the exterior surface of the Arch. The contractor developed the welding methods which were subsequently inspected by an independent laboratory using state of the art inspection methods.

34. How did the welding process change between original construction documents and the actual construction? What was the reason for the change? And how was the final solution developed?

The final welding techniques were developed by the contractor and the independent inspection laboratory.

35. What can you tell us about wrinkles in the stainless steel plates?

The stainless steel is very reflective. Any deformation becomes apparent. The spot welding on the inner surface of the outer steel skin has caused some small deformation of the surface of the polished stainless steel surface. The handling of the stainless steel during fabrication may have contributed to the surface condition which exists on the stainless steel surface. The surface condition was apparent at PDM's shop before shipping to St. Louis.

MF: I don't know entirely know what this photo is but I think it has something to do with, what you were talking about, the bolts connecting the two skins together.

BD: I'm not sure exactly what this is but it looks like some erection equipment. This is in the lower part of the Arch because you can see the major gap between the inner carbon steel skin and the outer skin and you can see the bolts. These bolts that you see here were tightened after the . . . oh, this is within the Arch itself. But you can see that they did have to tie these two things together, holding them together with these long rods. So, they did have to put supplementary supports in prior to the concrete being put in place.

MF: You were talking earlier about the welding that was going on. The welding was creating some dimples?

BD: Not this welding.

DS: You were, I think, talking about the bolts, right?

BD: Yes, as I recall, this is the outer skin. There is an angle. I guess it was put on like this. This was put in by electric type welding and that would create a dimple on the outside because of the heat of that [welding]. But then this was fastened to the other side and there was a bolt here, and this was tightened to compress the steel in this direction as opposed to the bars going in the other direction.

DS: Was this effectively the welding on the outside what you mentioned earlier you were concerned about?

BD: I was concerned. When I went there, I could see that. I could see the dimple from the heat. When you heat the metal up it contracts or expands and leaves the dimple that you can see. I was concerned about that.

DS: Was anything done to address it?

BD: No. They felt that that was the technology that we had and that we were going to have to accept that. I don't remember anybody in the Park Service or anybody else being particularly concerned about it.

DS: We can look at some current photos and see if you think it's still noticeable. But, what about the other pattern, the waviness, that you mentioned?

BD: When I first went to Pittsburgh/Des Moines Steel, I could see some waviness in the steel and I think, probably, it was caused by just the handling and the fabrication of the steel. When you handle stainless steel, turn it just a bit, you can put in some deformation that is minor but it's still, because it is a highly reflective surface, you can see it. I was concerned about that. I don't think there were others that were that concerned; there was nothing much you could do about it.

DS: Who was reviewing the samples, for example, the first pieces that were fabricated? Did you go out to look at them? Did the Park Service look at them?

BD: I remember going there alone doing it.

DS: So they didn't see it until it came to the site?

BD: No, I don't think so. There were two Bobs, we called them Bob and Re-Bob, who were with the NPS Construction Division. I think it was Bob Smith, but I cannot remember their last names, but they were more administrative type people. I was not administrative in that sense you know, big deal how high a building should be across the street, those were not issues. My primary job was to help develop the drawings and specifications and get this thing built. That was my . . .

DS: When you were going to the fabrication plant to look at the scale and when you were on site with the contractors and as the process moved along and then there was the hiatus of the structural stability question being raised and then it went back under construction again—did you feel that it was a fairly collegial atmosphere between the Park Service, the architect, and the various contractors. What was the character of the site meetings and discussions?

BD: I worked on many projects and I would say that the architect and the contractor would meet with the owner and we would have discussions. The three of us and the representatives of our consultants and the owner. I thought, you know, there is a tension in any building; there is a certain tension between the contractors and the owner and architect. Because, if issues come up, maybe the owner anticipated something, and I think that the Park Service handled it slightly different than other projects. George Hartzog was a very dominant force, representing the Park Service, and maybe he would lead the meetings and maybe make statements like "we will get back to you on that" or something or any questions they would handle more in a formal way.

DS: There were a lot of forceful people who were involved in this project, it sounds like.

BD: I think, because it was a unique project . . . I did not have a lot of meetings, I don't think, with MacDonald Construction Company. I remember Art Prichart, we would meet a few time, but they were very informal meeting. George Hartzog made sure that . . .

36. What do you recall any other differences between the plans and the actual construction methods? What was the reason for these differences?

Basically, the plans and specifications were followed. There were questions regarding the structure which our consultants Severud Elsted and Kruger addressed. The question regarding the cost of the inspection of the welds was addressed and settled during the construction period. There were questions regarding the interpretation of the specifications which were resolved and work proceeded.

DS: A lot of very creative people working together. Are there any other particular challenges during the design and construction that you recall? We talked a little bit about the stainless steel skin and the size of the plates changing.

BD: If you look at these original drawings, you see that, I don't remember how many, it could have been maybe ten panels going across the surface of the Arch. During the bidding process, I believe, the issue came up, could we use larger plates. If we could, we could cut down the number of welds. You know we had ten panels across, you can see the amount of welding that is involved. We wound up with three panels basically. Much fewer welds and this is all predicated on the size of panels which could be rolled, like glass and other materials, they get so big and there is no way of making them any bigger, so using the three panel system cut down the number of welds and, I think, simplified the construction a big deal. You could get a very consistent polish on these panels. I don't see a lot of variation. There is some variation between panels, but I think any time you are using building materials, there is always nature or whatever, there are always some little differences, they don't come out exactly. But, I think the polishing of the stainless steel is pretty uniform.

37. You were a part of the regular construction meetings at the site. What was the general attitude and rapport between the different companies and professions? How did the various entities work out discrepancies between construction documents, actual construction methods, and debates over structural stability?

I believe that there was great respect for Dr. Hannskarl Bandel's abilities, intelligence, in addressing technical issues that the contractor PDM raised during the construction period. I believe that he was fair and understood the implications of the contractor's questions. That respect that the contractor had for the engineers Severud Elstad and Kruger helped resolve issues raised during the construction period. The actual means and methods of construction are the contractor's responsibility, especially on this project with respect to the erection of the Arch.

38. Could you describe going up in the Arch during construction?

As the stairway was erected, one was able to walk up inside the Arch. The stairway followed closely behind the Arch erection. Aline Saarinen visited the Arch after the exterior was complete but before the stairway was completed. We climbed the stairway to near the top and she managed with her skirt to get to the top before the stair way and floor of the observation platform was in place.

I remember more the going to the erection platform on the outside of the Arch. The Arch was wider than the erection platform at the base of the structure. At higher elevations getting off the construction elevator on a small platform and climbing the ladder to the work platform was exciting as the ladder was not vertical so the one's view was looking down hundreds of feet to the ground below.

DS: You did tell us you were there when the Arch was nearing completion and that you stood on the top? Can you tell us about that?

BD: Well, as the Arch went up, there was this platform at the back of the Arch. The Arch is 54 feet on the base and as it gets to the top, it's 17 feet across the top on the external side of the Arch. So, there was a ladder you went up to this deck and—actually, there were two ways to get up. As the Arch went up, you had to go up a small elevator that the contractor had built so that they could get workmen up to the platform. As the Arch went up, of course it got narrower, so when you got off the elevator in the lower part of the Arch, the Arch was under you. But, when you got up higher, the ladder was on the outside of the Arch. When you are going up, you were looking down as you are going up to the platform. Once on the top of the platform, you could walk right off of the top of the Arch because the slope wasn't that much at that point. But, before this last piece was put in and before the legs were jacked a part, I would say they were about 24 inches a part. So, I put my foot on one side and one on the other side and you could feel the two legs of the Arch working independently. You could feel the movement of the two legs independently. So, that was a unique time. I don't know why I did it—by then I had three kids at home.

MF: Did they wear any harnesses or safety belts?

BD: No, no safety belts or harnesses or anything like that. You were just up there. There was a safety net under it as you can see. Maybe you would have only fallen 50 or 75 feet.

MF: I have right here a photo of the first section and I'm just wondering what thoughts went through your mind on this particular day.

DS: Were you on site?

BD: What happened was that Severud, Elsted, and Kreuger had Bob Moore on the site and we had a representative there, Ted Rennison. I was asked to come to the site, frequently in fact, probably every other week, I would go there for a day or two. This was to answer any questions that came up and carry information back to the office regarding the process. I wasn't there necessarily at critical times.

Structural and Stability Issues

39. In 1964, two consultants to PDM put a stop order on the Arch, saying that the stainless steel plates would buckle when the legs were jacked apart for the final section. They questioned the basic design of the Arch. The Bureau of Public Roads performed seismograph tests and agreed with PDM. What are your memories of this controversy?

I do not remember PDM's consultants putting a stop order on the construction of the Arch or the Bureau of Public Roads issue regarding seismograph tests. I do recall the Bureau of Reclamation meeting in Denver, Colorado, regarding the stability of the Arch. The bottom line was that the National Park Service had the confidence in the Saarinen office and Severud Elstad and Kruger, and therefore the erection work proceeded despite the concerns of other government departments. I recall meetings where a military officer ordered us to stop the work. There was no discussion just a direct order to do so, as I recall.

40. What can you tell us about the doubts engineers had about the orthotropic design and the fight between the Bureau of Reclamation experts and Severud, Elstad, and Kruger over structural stability of the Arch?

See answer to question 39.

41. Did you have doubts about the stability of the Arch?

No.

42. Did you have doubts about the Arch being finished?

No.

DS: How do you think these various engineers felt about the Arch? Do you think they thought here is a designer handing us a problem or do you think they thought it is a brilliant design and a great challenge or some of both? How did they feel about it as a design?

BD: Other engineers?

DS: Yes

BD: I think the fact that it was just a unique form in the first place, it was not what other engineers were building. You think of post and beam type construction and this being so unique and the fact that they were going to make the skin of the Arch, actually try to make it a part of the structure and not the initial . . . we looked at some drawings in the file here at Yale and it shows a sort of steel frame. I think many engineers might have thought of doing it that way. But coming up with this orthotropic plate design and using the post tension concrete and not only tensioning the concrete in the vertical direction but also in the horizontal direction with these bolts that go through the walls of the Arch and tightening those bolts so it compressed the concrete in two directions, really. Which I think was unique, because I worked on several buildings since then and we have used post-tensioned concrete for beams. The building here in New Haven, the Knights of Columbus building, we used post-tensioned concrete there to offset the loads that were on the cylindrical towers of that building. I think the other engineers really felt that it was unique. That brings me to the fact that there was this dispute that the Arch . . . was part way up and . . .

DS: The dispute about the structural stability?

BD: Yes, the structural stability, the . . . I don't know quite how it all happened, but the Bureau of Reclamation got into appraising the structural stability of the Arch, maybe because of questions raised by Pittsburg-Des Moines Steel looking for a contractor for erecting the Arch, the Bureau of Reclamation called us to Denver. John Dinkeloo, Hannskarl Bandel, and myself went to Denver and I remember this man was in a uniform, whether it was a military uniform, I guess it was. He just told us, there was really no discussion. He just said stop building the Arch because this is not going to work. He just said, "Stop!" This put the Park Service in a tough position as I recall. What do they do now? Do they proceed with the Arch or do they abandon it because of what the Bureau of Reclamation was saying. I think that George Hartzog and the Park Service decided to proceed having

faith in the Saarinen and the Severud, Elsted, and Kreuger office to proceed with the Arch because they definitely felt that it would.

MF: How did you feel about the structural stability?

BD: I had so much faith in it. I think at the time, because I had spent so much time with Hannskarl, I really thought that was fine. When you look at some of these photographs and see the amount of steel and material that went into it. It is a very unique design but you can see that it is well built.

MF: So it was a little bit of a shock to you when they requested the stop order, I'm sure.

BD: Right. Especially when you see all this erection equipment on the back of the Arch and the contractor was making measurements as to exactly where the Arch was and where the Arch was predicted to be and it was within tolerance, certainly all the way up. So it wasn't something that was starting to sag or fail in any way. I felt comfortable.

DS: Do you have any thoughts about where the objection came from? It originated, I think, from Pittsburgh/Des Moines Steel.

BD: I think so and the Bureau of Roads or something got into it, I think. It was a tough time, for me personally. I wasn't a partner in the firm or anything like that but on the other hand I was so involved in it and I did feel concerned.

DS: Did you feel concerned that it would actually stop the process?

BD: I was concerned that they would not have the courage to proceed. Not that I was concerned about the safety or structural stability of the Arch.

43. After completion of the Arch form, there was approximately a year of work involving the dismantling of the creeper derrick, cleaning, plugging holes, and construction of the tram. The Arch was structurally stable but there was still a lot of work to be done. What do you recall about this period? What were the issues of concern?

None that I recall about this period. The shop drawings for the conveyance systems were being submitted for our review prior the fabrication and installation in the Arch. We hired a mechanical and structural engineer, Spiegel and Zamecnik, to review the shop drawings prepared by the contractor, for structural capacity of all the conveyance system components.

Other Questions

44. Were there any incidents you could tell us about that were unusual or amusing?

There was a party in St. Louis; all men involved in the construction were invited. One man brought his wife because he said that his wife wanted to meet the men involved in this erection. When Joe Lacy, a partner of Eero, was asked to make a brief statement as the representative of our office, he said that as architects that our office is less interested in the erection but more interested in the conception.

45. What do you recall about Ted Rennison? What was his role in the project? What sort of personality was he?

Ted was our on-site representative working directly with the contractor and the Park Service there in St. Louis. He followed the day to day work on the site and wrote daily activity reports. He was very watchful of the contractor's workmanship. He performed well as our representative.

DS: So, as the project proceeded, your role was sort of as project architect but Ted Rennison had the day-to-day responsibility on site?

BD: Ted would write up daily reports of the progress of the work. He reported to me and we talked on a daily basis, frequently at least. Ted was somewhat confrontational, I would say.

DS: We have seen a lot of his papers from the project. There is quite a big collection down in the archives at the Arch.

BD: He was a little blunt. I don't know if I should say that or not, John Dinkeloo was the partner in charge. John, not particularly on this job, but John told me that I was never going to get anywhere because I was not tough enough. John almost got in a fight with one of the . . . I think Mr. Gregg, because, you know, he was so pushy, so dynamic. John really had the drive; I think I was a little more . . . I definitely wanted things done the way we specified and had drawn it and so on. I know on another project, I was called being too tough so it is always tough to know exactly . . .

DS: Perhaps it was good that you were more diplomatic on this project with so many other people that were not so diplomatic.

BD: With all these strong personalities and gifted people too when you think of Eero and his fantastic ideas and his ability to think about things and do them over and over and over again until he got them just the way he wanted. I mean, it seems like it never was sure when you made a drawing whether this was going to be the final one. Sometimes you just had to get the eraser out and do it all over again. We did this all by hand. This was before computers.

DS: When was the last time you were at the Arch?

BD: They did the *Good Morning, America* program with Bob Moore and myself in November of 1995. That's the last time I was there.